THE DESIGN AND CONSTRUCTION OF A PIEZOBLADE AND AN EVALUATION OF THE MARCHETTI DILATOMETER IN SOME FLORIDA SOILS

By

Alireza Boghrat

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To my dear children Pedram and Parastu

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TABLE OF CONTENTS

Page

ACKNOWLEDGMENT.................................. iii

LIST OF TABLES.................................. ix

LIST OF FIGURES................................. xi

ABSTRACT.. xvi

CHAPTER

1 INTRODUCTION........................... 1

1.1 Background........................... 1
1.2 Purpose.............................. 2
1.3 Scope................................ 3

2 DESIGN OF PIEZOBLADE........................ 4

2.1 Introduction............................ 4
2.2 The Blade............................... 5
2.3 The Transducer and Connector............ 7
2.4 The Cable............................... 20
2.5 Control and Recording Units............. 22
2.6 Calibration of the Pore Pressure Transducer.............................. 26
2.7 De-airing the Piezoblade................ 30

3 OTHER INSITU TEST DEVICES.................. 42

3.1 Introduction............................ 42
3.2 The Dilatometer Test.................... 42

3.2.1 The Equipment..................... 42
3.2.2 Dilatometer Test Procedure....... 44
3.2.3 Dilatometer Data Reduction....... 46

3.3 The Cone Penetration Test............... 51

3.3.1 The Equipment and Test Procedure. 51

Page

3.3.2 Cone Penetration Test Correlations 53

4 FIELD TESTING.............................. 56

4.1 Introduction.......................... 56
4.2 Test Site............................. 56

4.2.1 Location........................ 56
4.2.2 Site Geology.................... 57
4.2.3 Site Plans...................... 64

4.3 Piezoblade Testing.................... 64

4.3.1 Test Preparation................ 64
4.3.2 Types of Piezoblade Tests....... 70

4.4 Dilatometer Testing................... 71
4.5 Cone Penetration Testing.............. 73

5 LABORATORY SOIL TESTS..................... 77

5.1 Introduction.......................... 77
5.2 Sampling Procedures................... 77
5.3 Tests on Disturbed Samples............ 78
5.4 Tests on Undisturbed Samples.......... 81

5.4.1 Consolidation Tests............. 81
5.4.2 Triaxial Tests.................. 88

6 STEREO PHOTOGRAPHY STUDY.................. 96

6.1 Introduction.......................... 96
6.2 Equipment............................. 97

6.2.1 Container, Probe and Sand....... 102
6.2.2 Camera and Film................. 109

6.3 Testing Procedures.................... 112

6.3.1 Sand Placement.................. 112
6.3.2 Blade Penetration............... 116
6.3.3 Parallax Measurements........... 116

6.4 Scale Determination, Displacements and Strains........................... 120

6.4.1 Scales.......................... 120
6.4.2 Displacement.................... 125
6.4.3 Volumetric Strain............... 125

Page

6.5 Analysis of Stereo Photography.......... 126

6.5.1 Very Loose Sand.................. 126
6.5.2 Loose Sand....................... 128
6.5.3 Medium Sand...................... 128
6.5.4 Dense Sand....................... 131
6.5.5 Very Dense Sand.................. 133

6.6 Comparison of the Dilatometer and Cone Penetration Test Probes................. 133

7 ANALYSIS AND DISCUSSION OF RESULTS........... 137

7.1 Introduction............................ 137
7.2 Piezoblade Results...................... 137

7.2.1 Identification of High Drainage Layers.......................... 138
7.2.2 Site Uniformity.................. 139
7.2.3 Excess Pore Pressure Dissipation Versus Soil Type................ 143
7.2.4 Initial Excess Pore Pressure Versus OCR and I_D................ 152
7.2.5 Comparison of Piezoblade and Piezocone......................... 164

7.3 Dilatometer Test Results................ 166

7.3.1 Soil Classification.............. 166
7.3.2 Overconsolidation Ratio.......... 171
7.3.3 Constrained Modulus.............. 177
7.3.4 Marchetti Modulus................ 179

8 SUMMARY RESULTS AND SUGGESTIONS FOR FUTURE RESEARCH........................... 184

8.1 Summary................................. 184
8.2 Results................................. 186
8.3 Recommendation for Future Work.......... 188

APPENDIX

A PLOTS OF PIEZOBLADE TEST RESULTS............. 192

B RESULTS OF DILATOMETER TESTS.................. 210

C CONE PENETRATION TEST LOGS................... 233

D CONSOLIDATION TEST RESULTS................... 246

Page

E TRIAXIAL TEST RESULTS........................ 271

F STEREO PHOTOGRAPHY........................... 289

LIST OF REFERENCES................................. 310

BIOGRAPHICAL SKETCH................................ 312

LIST OF TABLES

Table Page

3-1. Values of f_s/q_c for Loose Medium and Dense Sands.................................. 55

4-1. List of Piezoblade Tests Performed......... 72

4-2. List of Dilatometer Tests Performed........ 74

4-3. List of Cone Penetration Test Soundings Performed................................. 76

5-1. List of Undisturbed Samples Taken.......... 80

5-2. Results of Tests on Disturbed Samples from Boring DS-1............................... 82

5-3. Results of Tests on Disturbed Samples from Boring DS-2............................... 83

5-4. Results of Tests on Disturbed Samples from Boring DS-3............................... 84

5-5. Results of Tests on Disturbed Samples from Boring DS-4............................... 85

5-6. Results of Tests on Disturbed Samples from Boring DS-5............................... 86

5-7. Consolidation Test Results................. 90

5-8. Triaxial Test Results...................... 93

5-9. Laboratory Results from Undisturbed Samples. 94

7-1. Percent Dissipation of Excess Pore Pressure in PBT After One Minute.................... 144

7-2. Penetration Excess Pore Pressures in Different Soils............................. 153

Table | Page

7-3. Soil Classification by Laboratory Dilatometer and Cone Penetration Testing.......... 168

7-4. Soil Classification by Laboratory Dilatometer and Cone Penetration Testing.......... 170

7-5. OCR's from Oedometer Dilatometer Testing... 174

7-6. Constrained Moduli (M), from Oedometer and Dilatometer Testing........................ 178

7-7. Calculation of Dilatometer Modulus E_D from CU Triaxial Tests.......................... 180

7-8. Comparison of E_D Values from DMT and CU Triaxial Tests............................ 182

LIST OF FIGURES

Figure Page

2-1. Sketch Showing Piezoblade Dimensions........ 6

2-2. Dimensions of the EPN-0350-100 Pressure Transducer................................ 8

2-3. Side View of the Piezoblade Probe........... 9

2-4. Front View of Piezoblade Probe.............. 10

2-5. Location of the Transducer in the Piezoblade. 11

2-6. Piezoblade Transducer Holder................. 12

2-7. Piezoblade Showing the Transducer Cavity and Wiring Groove............................. 13

2-8. The Transducer in Place in the Piezoblade.... 13

2-9. Piezoblade Porous Stone Holder.............. 15

2-10. Front of the Piezoblade with the Porous Stone and Its Holder................................ 16

2-11. Detail of Porous Stone Holder............... 16

2-12. The Piezoblade Connector in Place with the Transducer and Cable Wires Attached.......... 17

2-13. The Piezoblade Electrical Connector.......... 18

2-14. Special Tool to Screw the Connector in Place. 19

2-15. The Piezoblade Cable........................ 19

2-16. The De-airing Unit Holder................... 21

2-17. The Test Control and Recording Unit......... 21

2-18. The Clamp Box............................... 23

2-19. The Control Box Circuitry................... 24

Figure Page

2-20. The Test Control Box.......................... 25

2-21. The Voltmeter................................. 25

2-22. The Linear Instrument Corporation Recorder. 27

2-23. The Calibration Unit.......................... 28

2-24. Photograph of the Calibration Unit......... 29

2-25. Calibration Curves for Different Excitation Voltages....................................... 31

2-26. The De-airing Unit Holder................... 34

2-27. The De-airing Unit Cylinder................. 35

2-28. The De-airing Unit Top Section............. 37

2-29. The De-airing Unit Screen................... 38

2-30. The Assembled De-airing Unit............... 39

2-31. Photographs of the De-airing Unit's Cylinder, Top and Screen..................... 40

2-32. Photograph of the De-airing Unit and the Piezoblade................................... 41

3-1. The Dilatometer Test Equipment............. 43

3-2. The Flat-blade Dilatometer.................. 43

3-3. The Control Unit.............................. 45

3-4. The Calibration Equipment................... 45

3-5. Dilatometer Soil and Density Evaluation Chart.. 49

3-6. Begemann Tips................................. 52

3-7. Cone Penetration Test Soil Evaluation Chart. 54

4-1. Location of Alachua County and the City of Gainesville..................................... 58

4-2. Location of Lake Alice and Lake Wauberg..... 59

4-3. Topographic Map of Lake Alice............... 61

Figure Page

4-4. Topographic Map of Lake Wauberg........... 62

4-5. Block Perspective Diagram of Ocala Limestone Structure Contour Map............ 63

4-6. Site-1, North of Lake Alice................ 65

4-7. Site-2, East of Lake Alice................. 66

4-8. Site-3, West of Lake Alice................. 67

4-9. Site-4, North of Lake Alice................ 68

4-10. Site-5, Northwest of Lake Wauberg.......... 69

5-1. The Hand Auger Sampler..................... 79

5-2. The Swedish Fixed Piston Sampler........... 79

5-3. The Anteus Consolidometer.................. 87

5-4. The Soil Test (Model C-221) Consolidometer. 89

5-5. The Triaxial Test Apparatus................ 91

6-1. Stereo Image-Displaced Camera.............. 98

6-2. Stereo Image-Displaced Object.............. 99

6-3. General Two-Dimensional Displacement....... 100

6-4. Probe Penetrated in Sand................... 101

6-5. The Test Container......................... 103

6-6. Stainless Steel Dilatometer Probe.......... 104

6-7. 8/20 Edgar Sand............................ 105

6-8. 20/30 Edgar Sand........................... 106

6-9. 30/65 Edgar Sand........................... 107

6-10. Edgar Glass Sand........................... 108

6-11. Test Setup................................. 110

6-12. Double Projector Anaglyphic Plotter........ 111

6-13. Tracer Point-Platen Assembly............... 113

Figure | Page

6-14. Compaction Equipment. Left-Vibratory Compactor. Right-Constant Energy.......... 115

6-15. Jack and Extensions........................ 117

6-16. Grid Element Numbering System.............. 118

6-17. Node Point Numbering System................ 119

6-18. Camera--Model and Projector--Stereo Image Relationships............................ 121

6-19. Plots of Displacement and Volumetric Strain for Very Loose Sand....................... 127

6-20. Plots of Displacement and Volumetric Strain for Loose Sand............................ 129

6-21. Plots of Displacement and Volumetric Strain for Medium Sand........................... 130

6-22. Plots of Displacement and Volumetric Strain for Dense Sand............................ 132

6-23. Plots of Displacement and Volumetric Strain for Very Dense Sand....................... 134

7-1. Piezoblade Profiles Across Site-1.......... 140

7-2. Piezoblade Profiles Across Site-3.......... 141

7-3. Piezoblade Profiles Across Site-5.......... 142

7-4. Percent Dissipation of Excess Pore Pressure Versus I_D for Different Soils.............. 149

7-5. Percent Dissipation of Excess Pore Pressure Versus I_D for Different Soils (I_D = log scale)............................. 150

7-6. Penetration Excess Pore Pressure Versus OCR for Sand and Silty Sand................ 159

7-7. Penetration Excess Pore Pressure Versus OCR for Sandy Silt....................... 161

7-8. Penetration Excess Pore Pressure Versus OCR for Silt............................. 162

Figure Page

7-9. Penetration Excess Pore Pressure Versus OCR for Clayey Silt........................ 163

7-10. Penetration Excess Pore Pressure Versus OCR for Silty Clay and Clay................ 163

7-11. Plots of Excess Pore Pressure Versus Depth from Piezocone and Piezoblade Sounding at Site-1.. 165

7-12. Typical Profiles of K_D Versus $\overline{P}_v$ for Uncemented Cohesive Soil in Simple Unloading................................ 173

7-13. Correlations between OCR and K_D, Including Florida Data.............................. 176

7-14. Plots of E_D Versus ΔP..................... 183

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By

Alireza Boghrat

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Chairman: Dr. John L. Davidson
Major Department: Civil Engineering

The primary purpose of the research was to develop a better understanding of the Marchetti flat-plate dilatometer, especially with regard to the test drainage conditions and the experimentally derived correlations.

To evaluate the drainage conditions around the dilatometer, a new insitu testing device, identical in shape but measuring only pore water pressures, has been designed and constructed. It consists of a thin steel blade with a flush mounted porous cap and an internal pressure transducer. This piezoblade is pushed into the ground and the pore pressures measured and recorded on a strip chart recorder.

A program of field testing was carried out at five sites near the University of Florida campus. Standard

dilatometer tests and piezoblade soundings, in which the probe was penetrated in 20 cm increments with a one minute or more delay between steps, were performed adjacently. The incremental piezoblade insertion was to simulate the dilatometer test procedure. From an analysis of the sounding results, a plot was developed of percent dissipation of excess pore pressure versus soil type. This gives a quantitative measurement of the drainage conditions (drained, partially drained, undrained) during dilatometer testing in any soil.

In addition to the dilatometer and piezoblade soundings at each site, cone penetration tests were performed and disturbed and undisturbed samples recovered for laboratory testing. A comparison of the dilatometer and the laboratory test results allowed an evaluation of the Marchetti correlations which were developed for soils in Italy. It was found that the dilatometer very accurately identified the soils through which it was penetrating. The existing parameter correlations, however, overpredicted both the overconsolidation ratio and the constrained modulus and underpredicted the Young's modulus. Correlations better suited to these particular Florida soils were developed.

A laboratory study was performed to compare the disturbance effects surrounding penetrated blade and cone-shaped probes. This was carried out in dry sand using a stereo photography technique to accurately measure grain displacement in the vicinity of the probe. The dilatometer-shaped probe was found to cause much less disturbance of the soil than the standard cone penetration test tip.

CHAPTER 1

INTRODUCTION

1.1 Background

Soil is an integral part of most civil engineering structures. For many, soil provides the foundation support. Soil is used in many projects as a construction material, e.g., earth dams, fills, reclamation projects and in highways and airfields. Soil interaction with the structure is important, e.g., in tunnels, pipes, underground structures, retaining walls and excavations. Other soil behavior such as slope stability, shrink-swell potential, freeze-thaw properties, behavior during earthquake or vibrations or potential for the storage of industrial fluids may be important.

For any structure which is to be properly designed, the extent, variability and properties of the soil must be known. The traditional means of obtaining these has been to make a boring, recover samples and perform laboratory tests. In the past decade, however, there has been a rapidly growing interest within the geotechnical profession in the insitu determination of soil properties. In such a determination, a device is inserted into the ground, the test performed and the soil properties calculated, usually by means of an experimentally derived correlation. Some of the advantages of insitu testing are

1. They are performed in the natural environment and under natural conditions with the correct moisture and stress regimes.
2. Disturbance is much reduced since sampling, handling, transportation and specimen preparation are eliminated.
3. Compared to most laboratory tests, insitu testing time is shorter.
4. The cost of insitu testing is normally less, relative to the quantity of data acquired.

Many different insitu testing devices have been developed, often for the determination of a single soil parameter. A recently introduced piece of equipment, the Marchetti flat plate dilatometer, however, has the potential for determining a wide variety of geotechnical parameters. It has been widely used in Italy and has proved very successful and accurate in obtaining various strength, insitu stress and compressibility properties. The University of Florida Geotechnical group possesses the first Marchetti dilatometer in the United States.

1.2 Purpose

The primary purpose of this research is to develop a better understanding of the Marchetti dilatometer especially with regard to the test drainage conditions and the experimentally derived correlations.

1.3 Scope

In order to evaluate the drainage conditions around the Marchetti dilatometer, a new insitu testing probe, identical in shape but measuring only pore pressures, was developed. Associated calibration and de-airing devices were also designed and built.

A program of field testing at five sites near the University of Florida campus was carried out. Dilatometer , the new piezoblade and cone penetration test soundings were made and disturbed and undisturbed samples collected for laboratory testing.

Based on the test results conclusions are drawn regarding the dilatometer drainage conditions and the validity of the Marchetti correlations in the tested Florida soils.

CHAPTER 2

DESIGN OF PIEZOBLADE

2.1 Introduction

The piezoblade consists of a stainless steel body, the same shape and dimensions as the Marchetti dilatometer, but with a flush mounted pore pressure transducer in place of the expanding membrane. It is penetrated vertically into the ground at a slow constant rate using the University of Florida cone penetration test truck. The device can be used to record both the pore water pressures generated during insertion and the dissipation of excess pore pressures if penetration is ceased. During penetration, the magnitude and sign of the recorded pore pressures indicate both the soil type and the density or overconsolidation ratio state.

An advantage of the piezoblade is that its thin (14 mm) plate shape disturbs the soil less than the standard, 36 mm diameter, piezocones. This has been demonstrated in an experimental laboratory study described in detail in Chapter 6.

The purpose of building the piezoblade was not simply to develop a better, less disturbing, piezometric probe, but rather to have an instrument which would permit a better understanding of the Marchetti dilatometer.

Since the dilatometer itself does not measure pore pressures, it is not possible to know, except in the extreme and obvious cases, whether a test is being performed in a drained, undrained or partially drained manner. By performing piezoblade and dilatometer tests adjacently in the field, it will be possible to relate dilatometer parameters and drainage conditions.

2.2 The Blade

The body of the piezoblade is identical to that of the dilatometer, Figure 2-1. It was machined from a single piece of stainless steel. The male threads are three millimeters deep and three millimeters apart, and require a short adaptor to connect to the standard cone penetration test outer rods. The cavity for the pore pressure transducer and porous stone assembly, and the groove for the wiring and thermal compensation section, were machined into the blade. The original dilatometer shape was chosen by Dr. Marchetti. The width of 90 mm was chosen so that the dilatometer could, if necessary, be penetrated through standard 100 mm and 4 inch ID casing. The thickness of the blade, 14 mm, was chosen as small as possible consistent with the requirement that it not be easily damaged or bent.

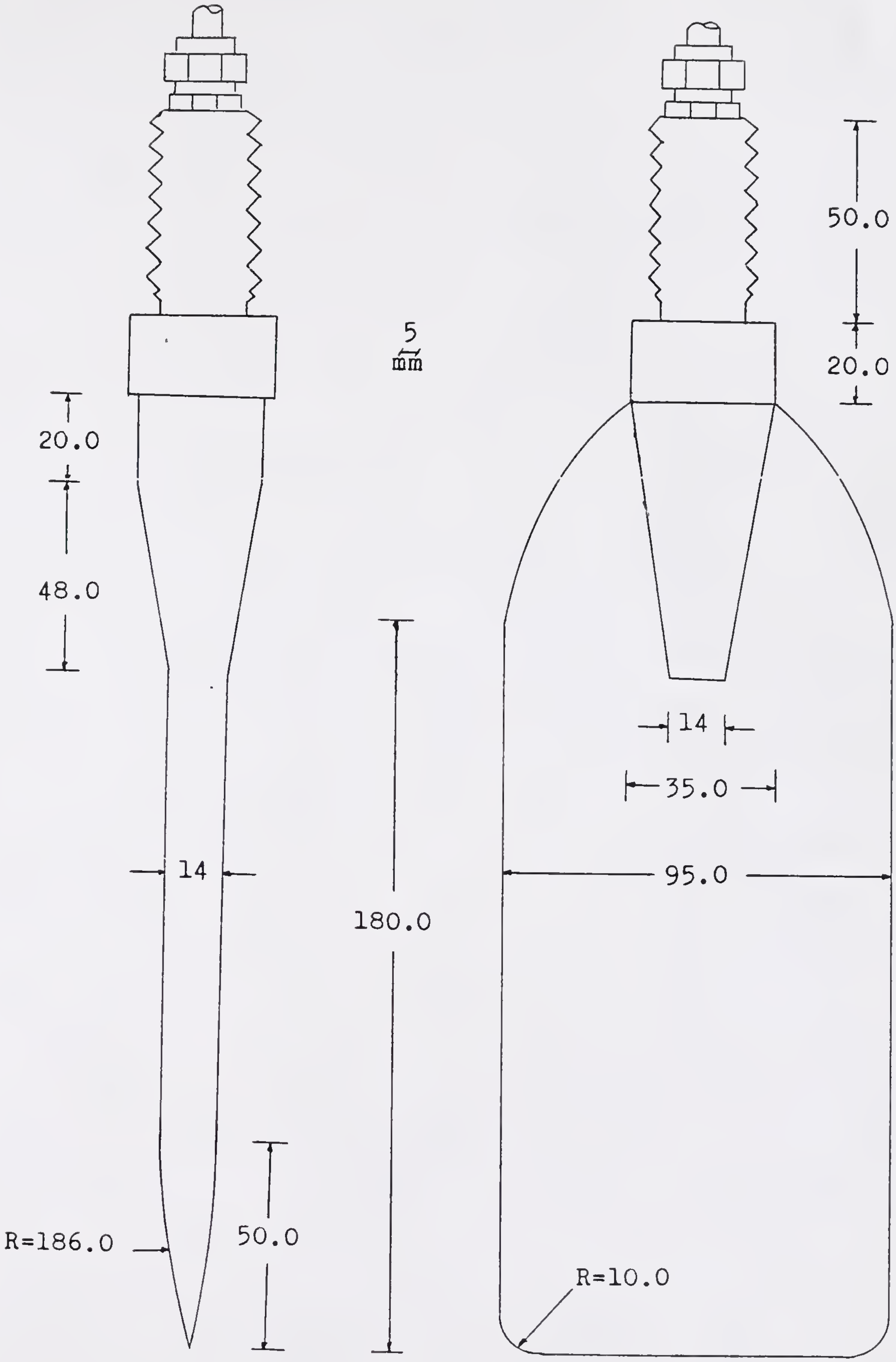


Figure 2-1. Sketch Showing Piezoblade Dimensions

2.3 The Transducer and Connector

The transducer is the main component of the piezoblade. It must have a high sensitivity in order to measure small changes in pressure and be as robust and well protected as possible. The transducer chosen was a model number EPN-0350-100 with a range of 0 to ±100 psi and an overrange of ±200 psi. The recommended excitation is 15 volts and the operating temperature range is $-60^{o}F$ to $+250^{o}F$. Figure 2-2 shows detailed dimensions of the transducer. The cable which connects the piezoblade to the surface control and recording instruments is 100 feet long. The voltage drop in the wire through the cable was measured and found to be 0.05 volts. Thus, to supply 15 volts to the transducer, the input voltage at the surface must be 15.05 volts. Output signals were measured at the transducer and after passing through the 100 foot wire. No change in signal was found at any level.

Front and side view sketches of the piezoblade with important dimensions are included in Figures 2-3 and 2-4. Figure 2-5 shows in detail the location of the transducer in the piezoblade. The transducer is permanently held in place by the holder shown below the transducer in Figure 2-5 and in detail in Figure 2-6. The holder is cemented in place using a special glue described later, and is removed only if the transducer is damaged and to be replaced. Figure 2-7 is a photograph of the piezoblade showing the transducer cavity and groove. In Figure 2-8 the transducer

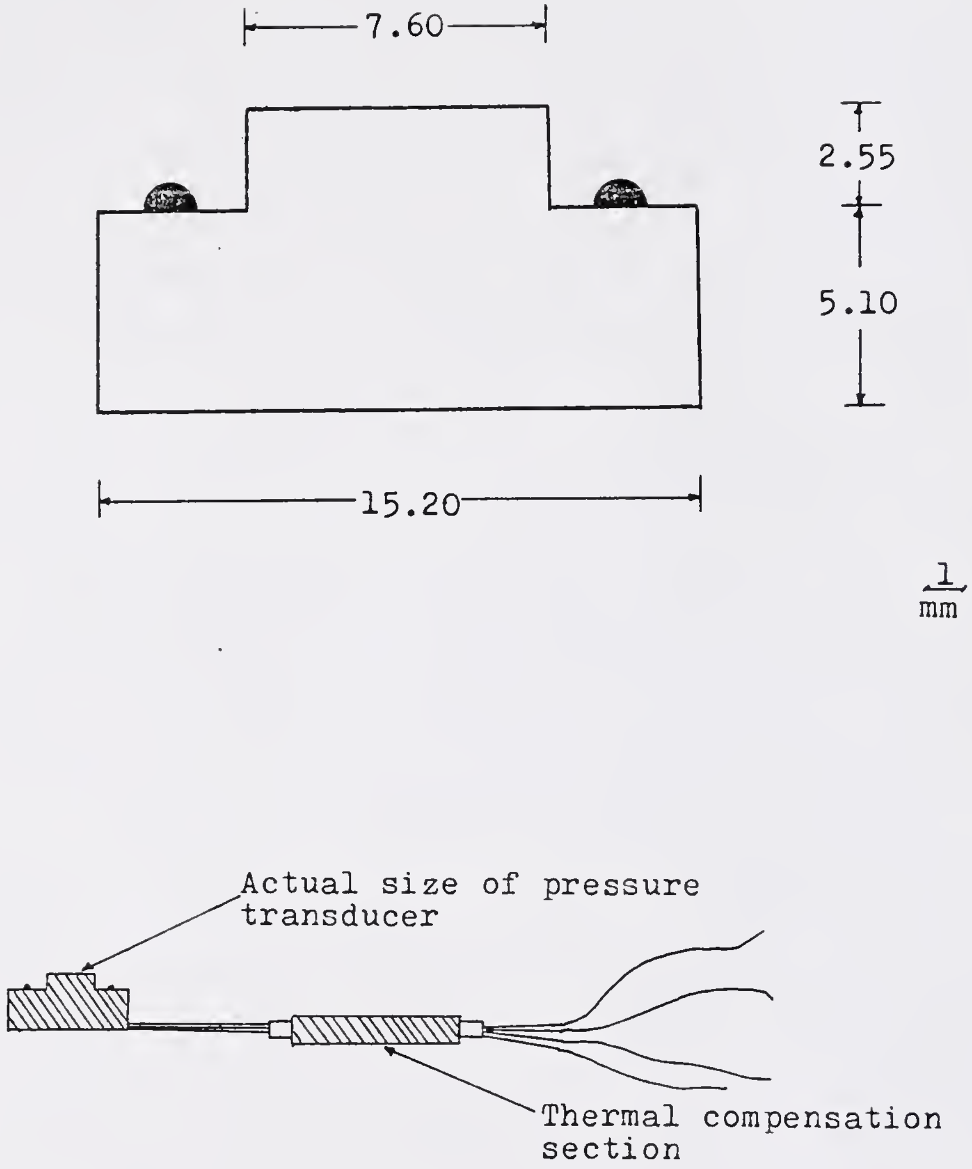

7.60
2.55
5.10
15.20
1
mm
Actual size of pressure transducer
Thermal compensation section

Figure 2-2. Dimensions of the EPN-0350-100 Pressure Transducer

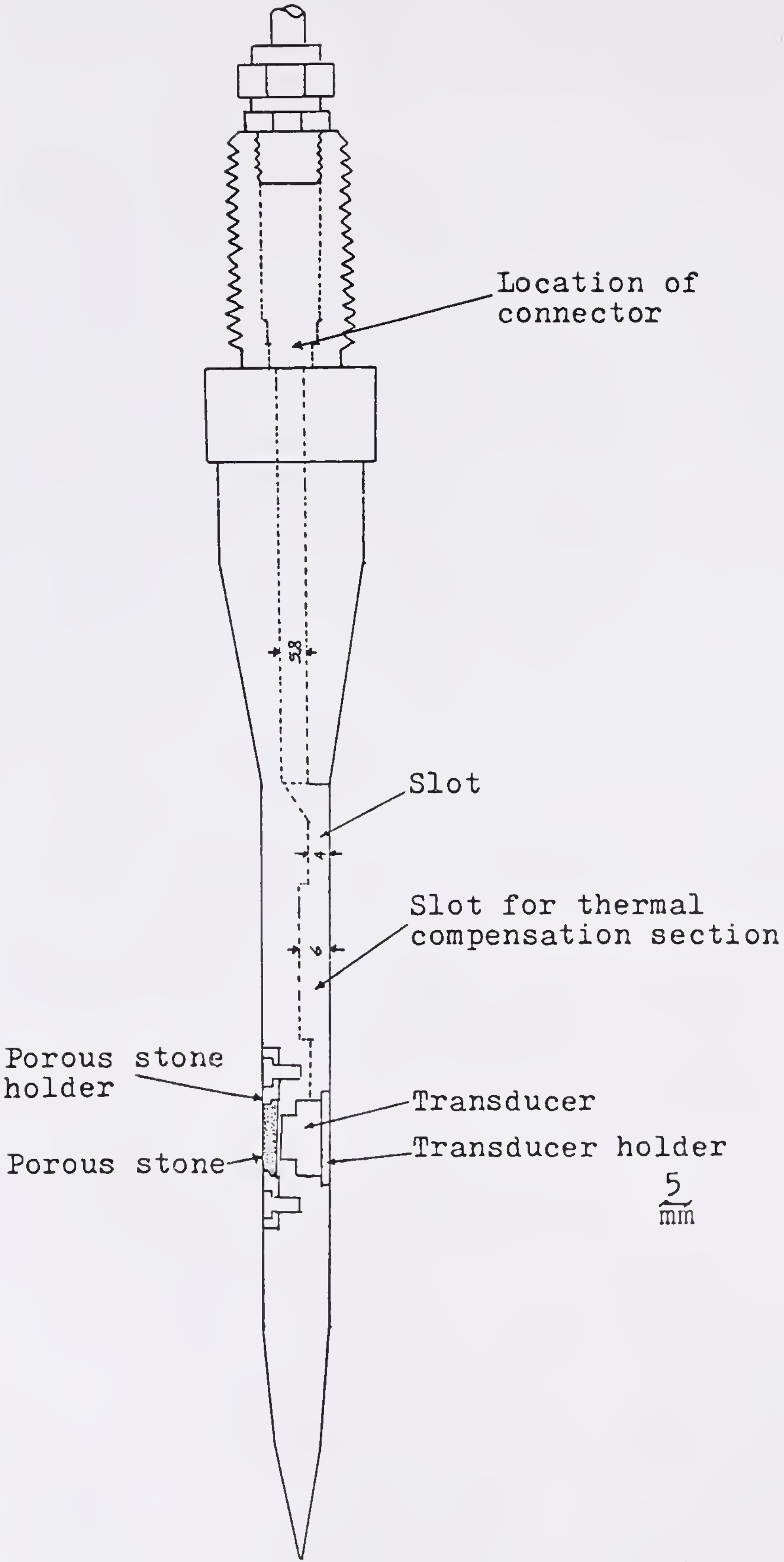


Figure 2-3. Side View of the Piezoblade Probe

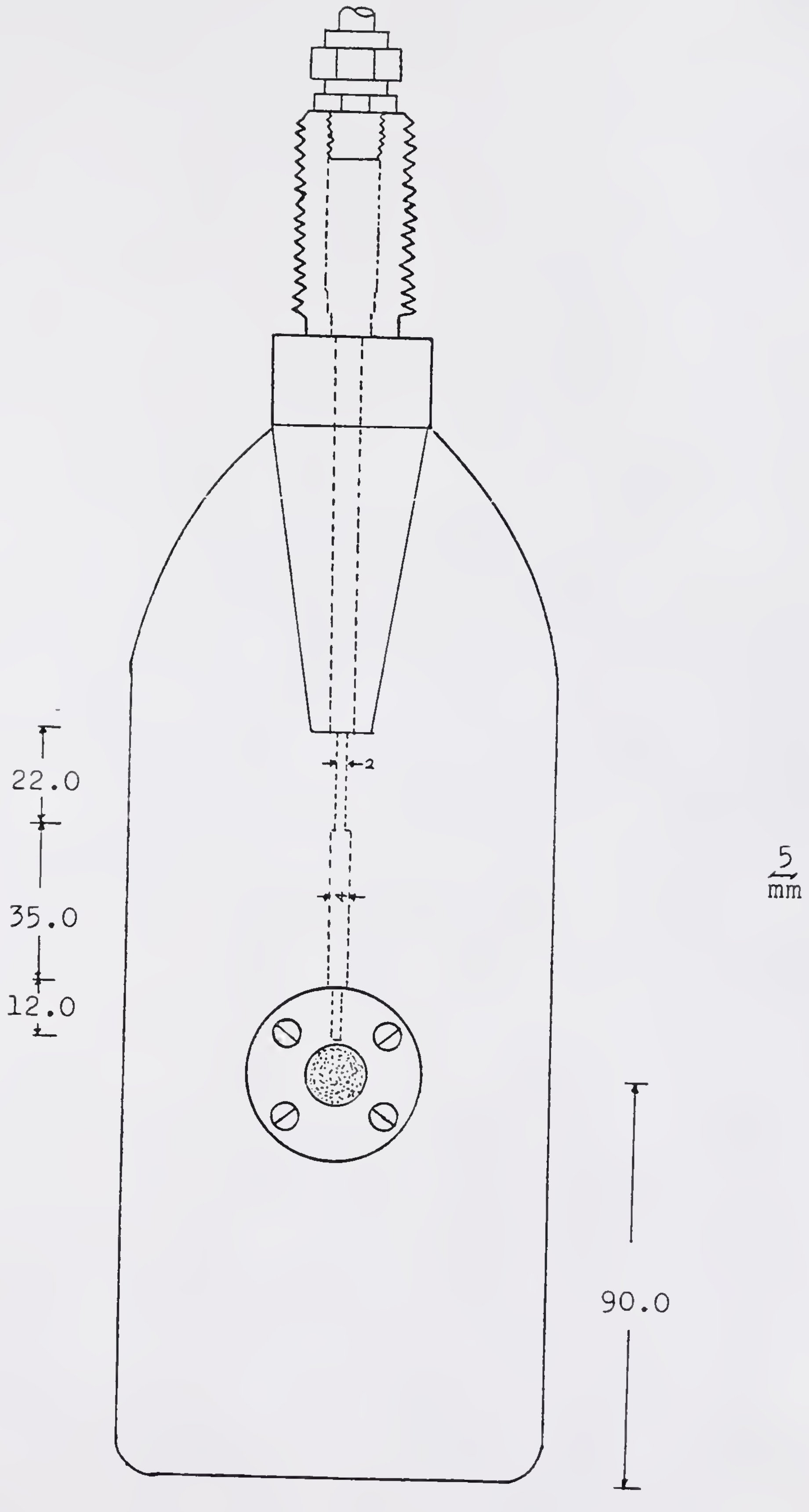


Figure 2-4. Front View of Piezoblade Probe

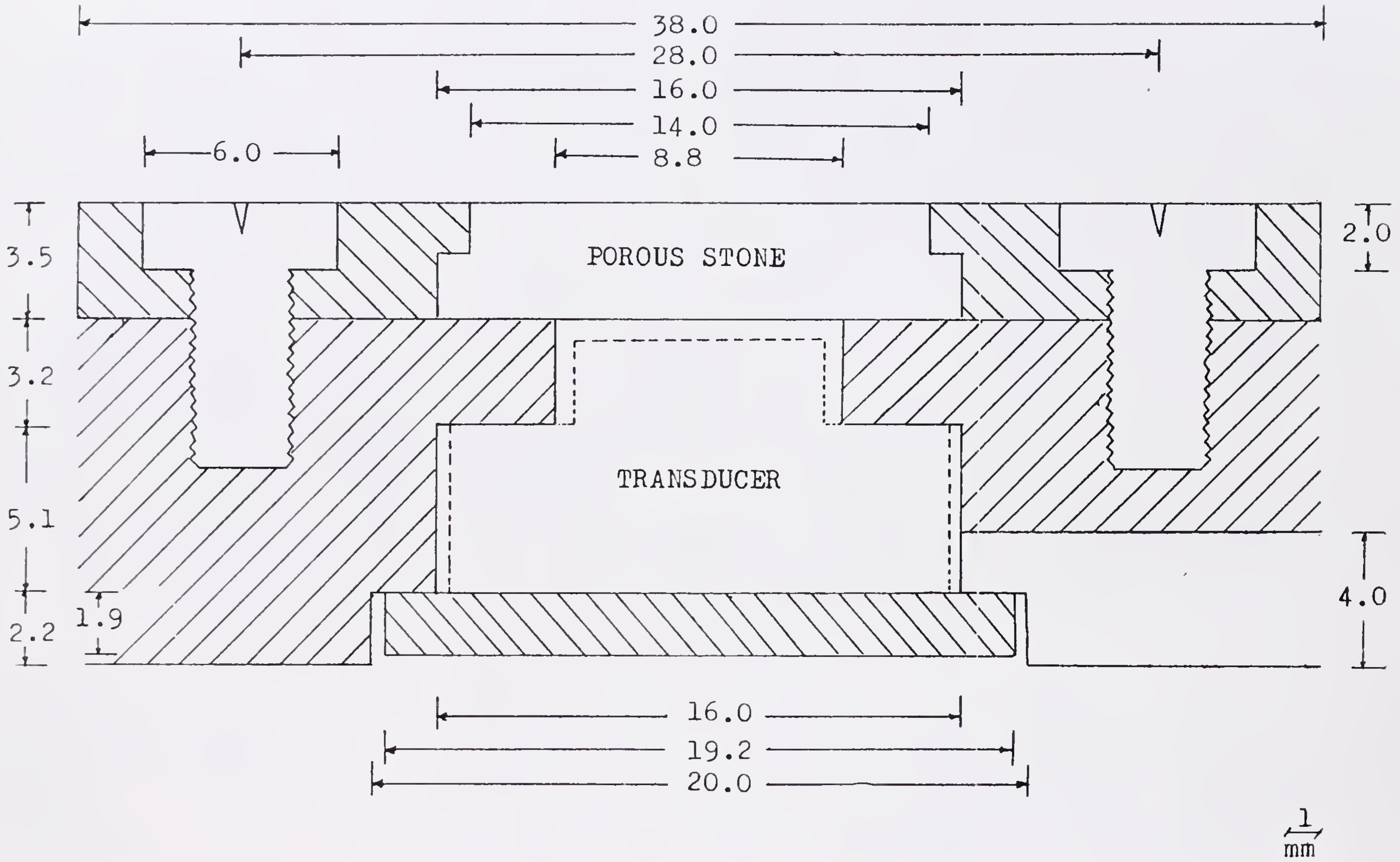


Figure 2-5. Location of the Transducer in the Piezoblade

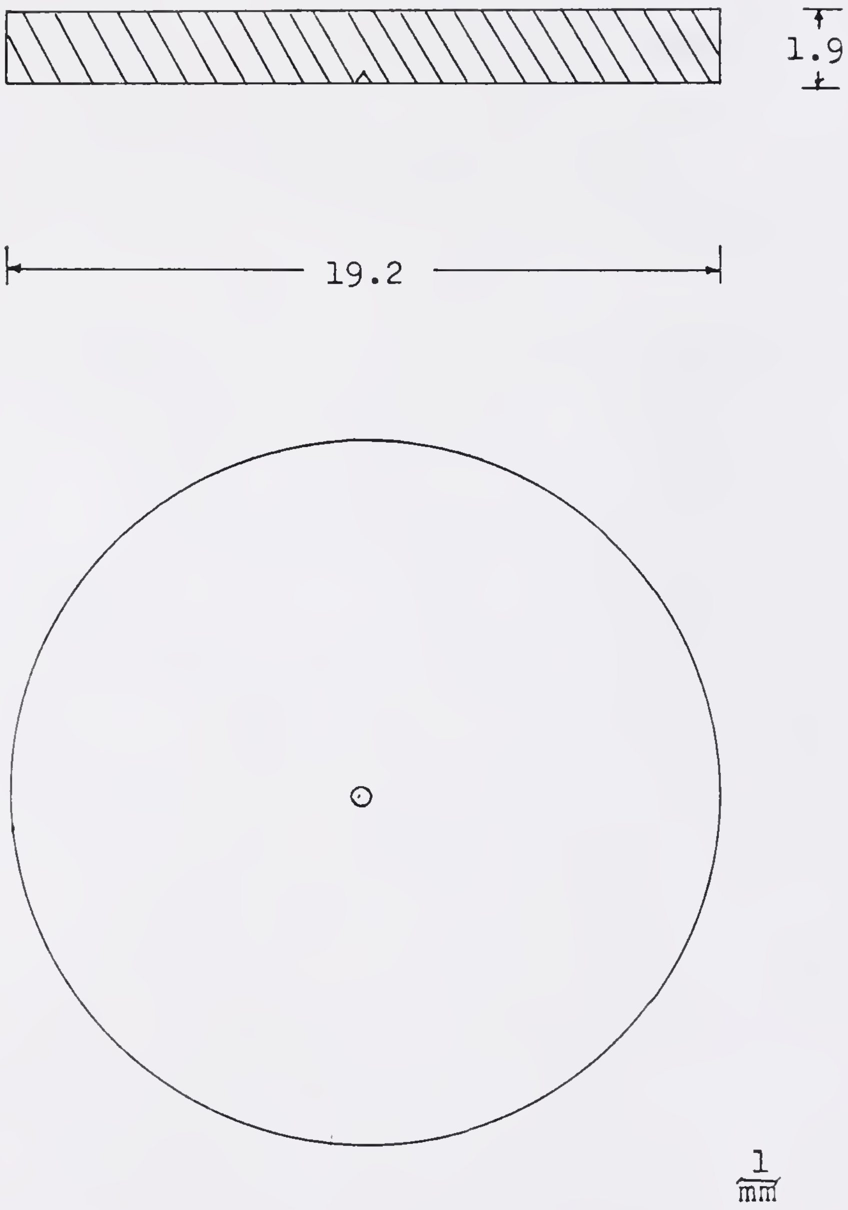


Figure 2-6. Piezoblade Transducer Holder

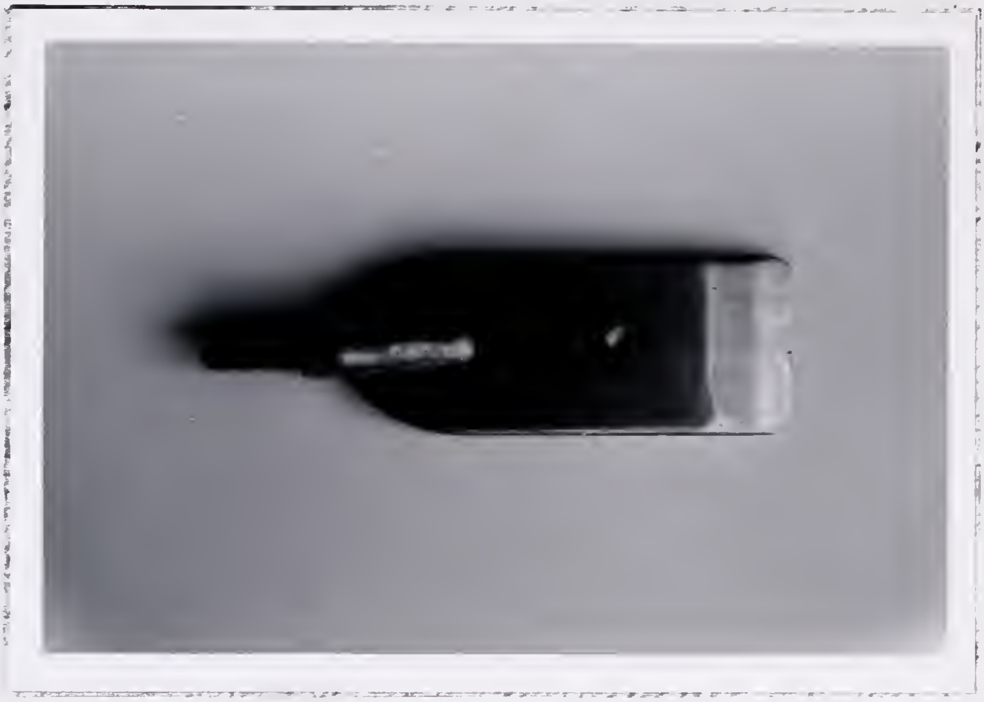


Figure 2-7. Piezoblade Showing the Transducer Cavity and Wiring Groove

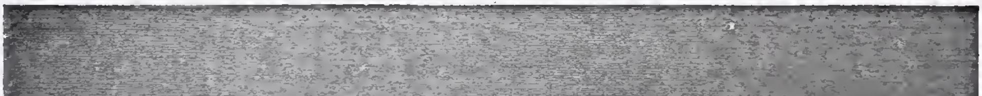
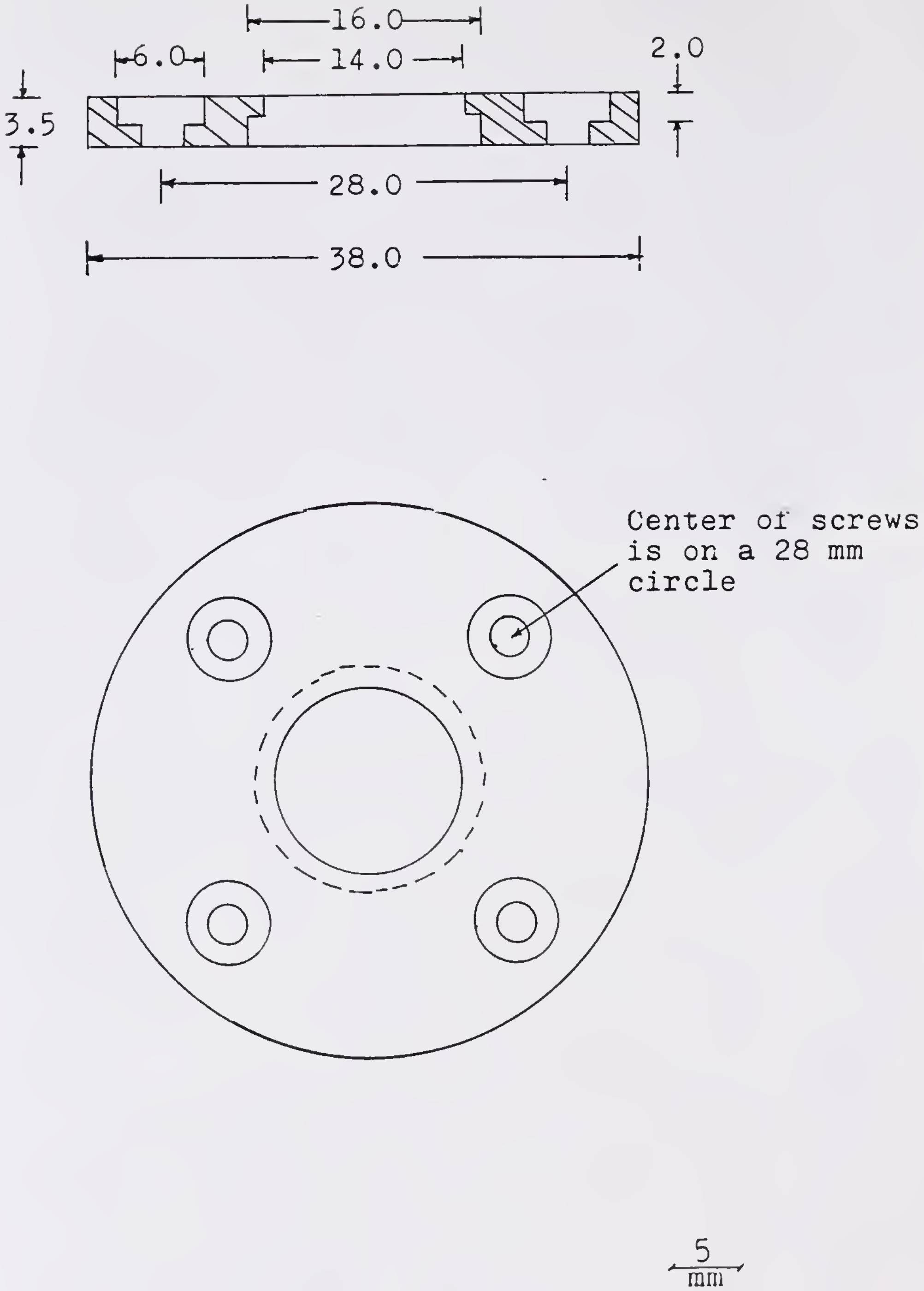


Figure 2-8. The Transducer in Place in the Piezoblade

is in place just prior to affixing the holder.

Located above the transducer in Figure 2-5 are the porous stone and its holder. The holder, shown in detail in Figure 2-9, consists of a stainless steel ring with counter-sunk screws by which it is attached to the blade. It holds the very fine porous stone in place. Photographs in Figures 2-10 and 2-11 illustrate this part of the piezoblade.

With the transducer in place, the wires lie in the groove and are threaded through the 5.8 mm hole to the head of the blade (see Figures 2-3 and 2-4). At this location a watertight connection is required between these wires and the 100 foot cable. A search of electrical supply companies failed to produce a connector of the desired size. The junction described below was therefore designed and has performed satisfactorily. Within the threaded head of the piezoblade, shown hatched in Figure 2-12, the plastic disc A with four pins protruding through each face is seated on an O-ring and held down by tightening the threaded ring B. Parts A and B are shown in detail in Figure 2-13. Ring B is screwed using the special tool shown in the photograph of Figure 2-14. The four wires from the transducer and the four from the cable, Figure 2-15, are soldered to the lower and upper pins respectively of the disc A before it is inserted. The end of the cable has a swagelok 1/4 inch male end which screws into the top of the piezoblade head making a watertight connection.



Center of screws is on a 28 mm circle

Figure 2-9. Piezoblade Porous Stone Holder

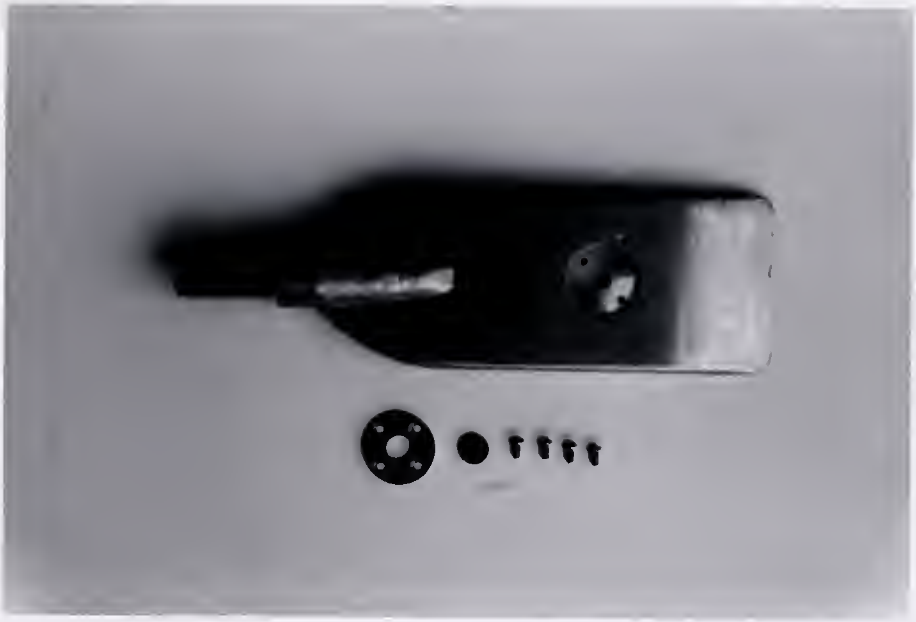


Figure 2-10. Front of the Piezoblade with the Porous Stone and its Holder

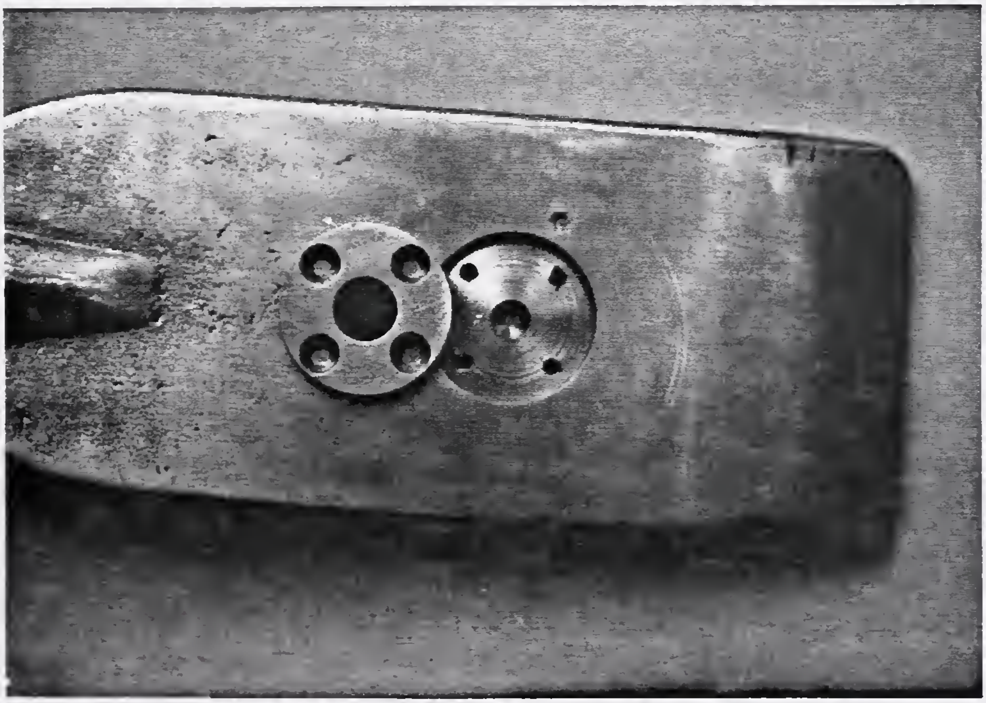


Figure 2-11. Detail of Porous Stone Holder

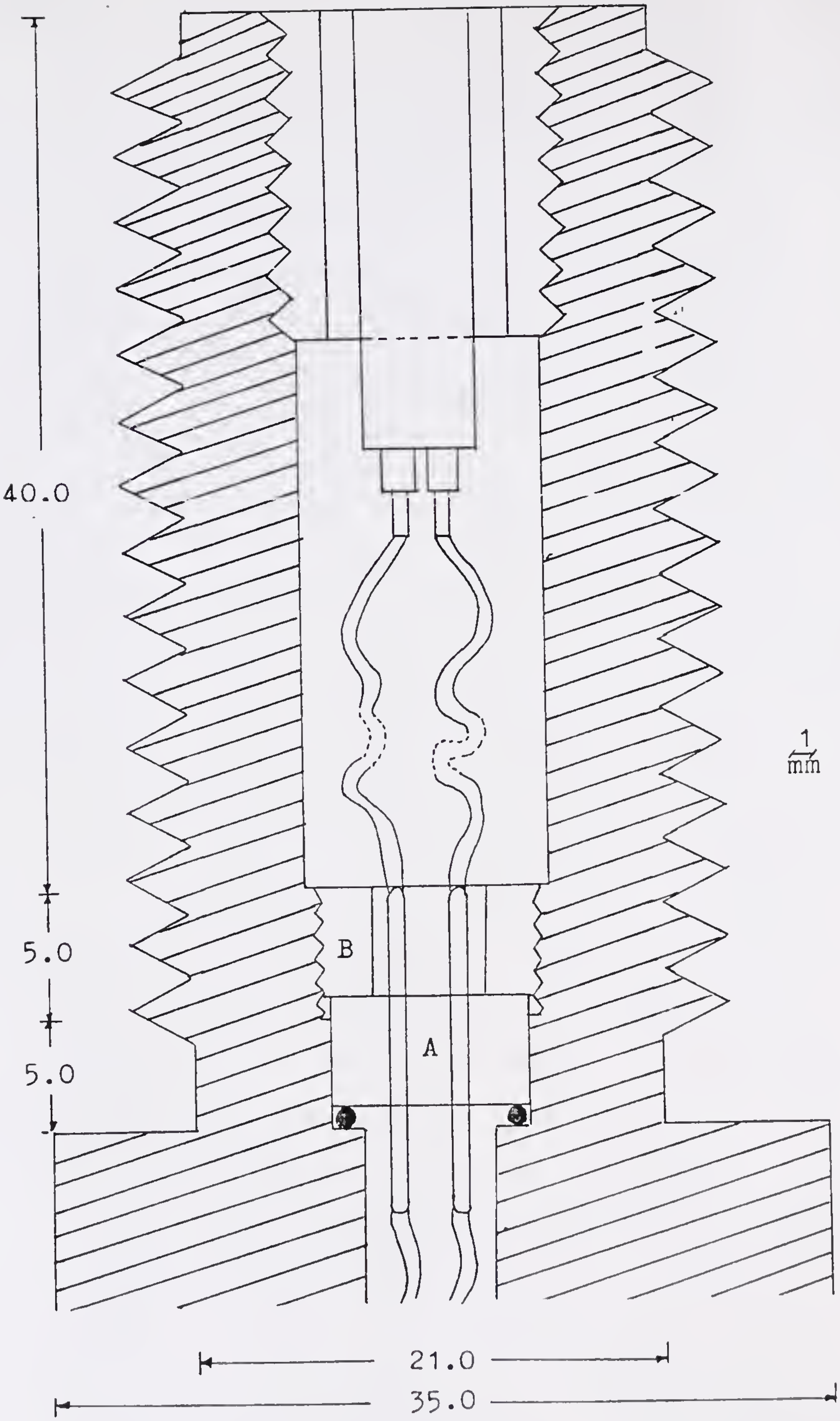


Figure 2-12. The Piezoblade Connector in Place with the Transducer and Cable Wires Attached

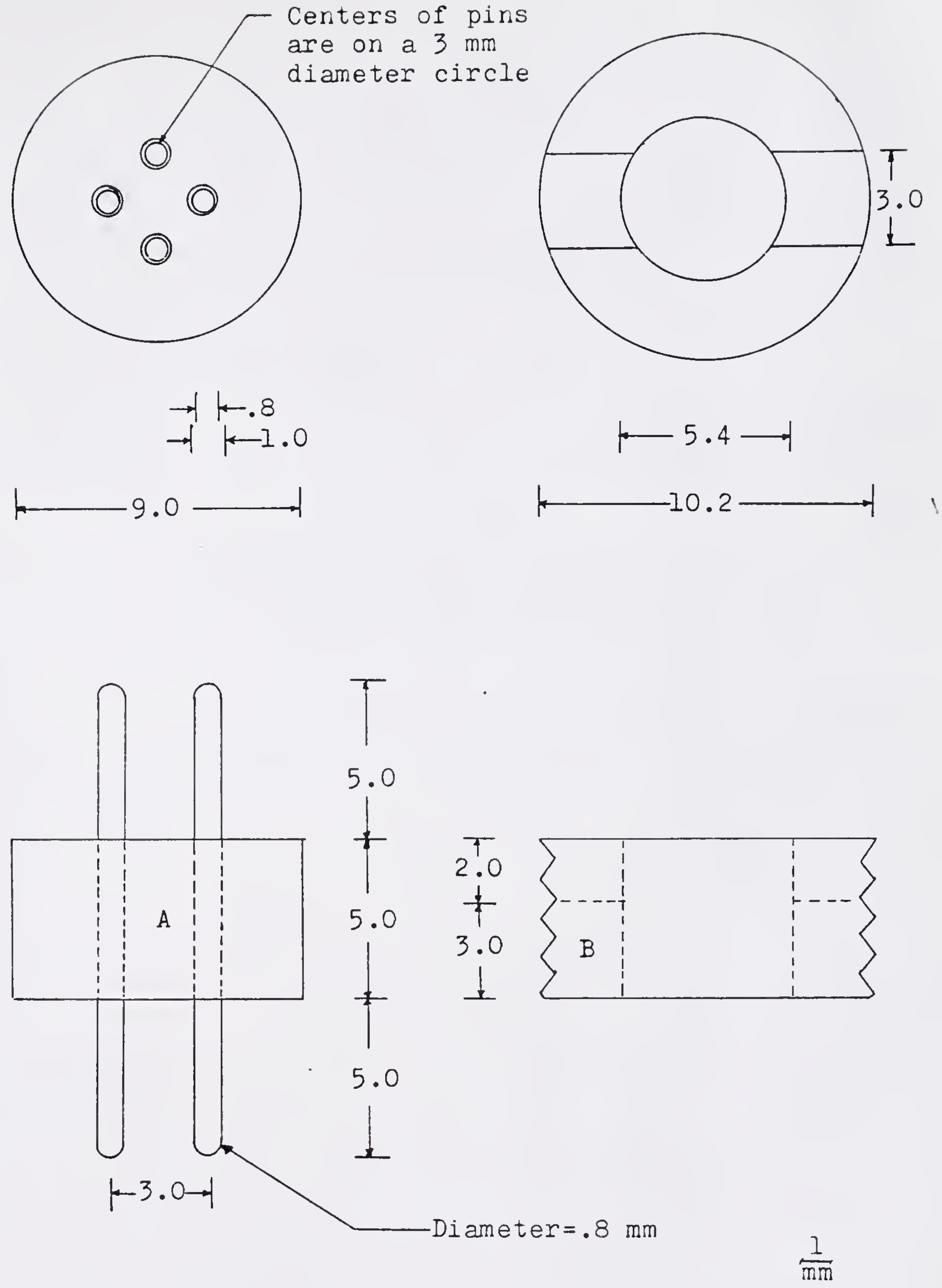


Figure 2-13. The Piezoblade Electrical Connector

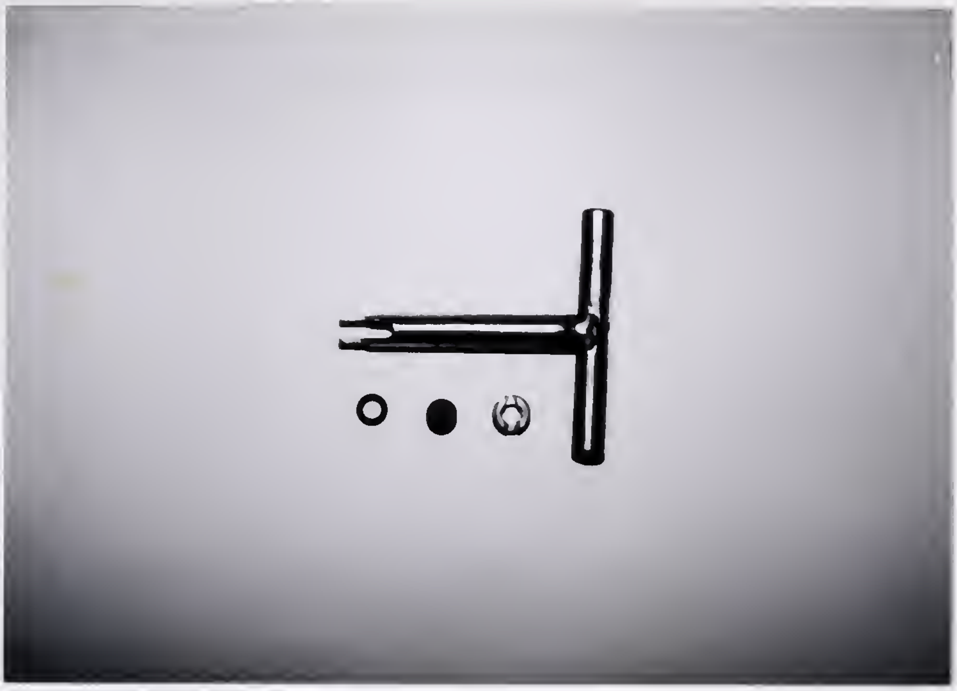


Figure 2-14. Special Tool to Screw the Connector in Place

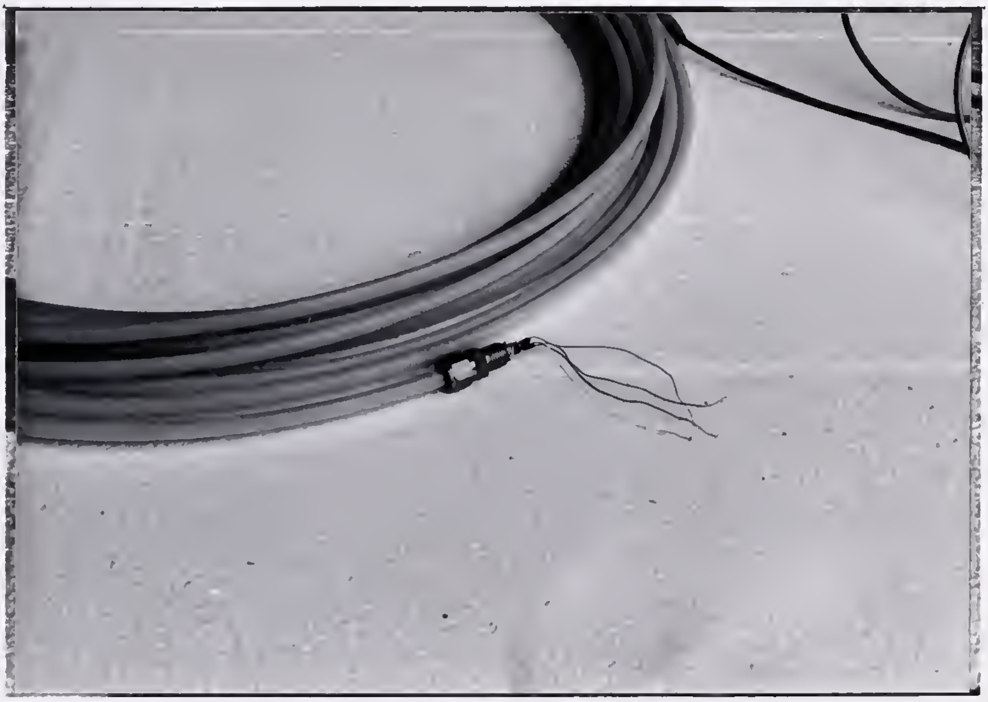


Figure 2-15. The Piezoblade Cable

With the transducer, the thermal section and the connector all in place, the transducer holder is attached. It is held temporarily in place by the de-airing unit holder, Figure 2-16. The center screw impinges on the indentation on the back of the transducer holder, Figure 2-6. The groove and the surroundings of the transducer holder are then covered with a special glue. This was made by mixing a few drops of Schnellklebstoff FFX60 with about a 1/4 teaspoon of the powder Schnellklebstoff FFX60 Komponete A. This produced a strong glue which, after a year of testing, remains intact.

2.4 The Cable

A 100 foot length of cable connects the piezoblade to the ground surface instrumentation. The cable, consisting of four conducting wires which transmit to and from the transducer, is enclosed in 9/32" ID, 3/8" OD plastic tubing. The cable was threaded through the tubing using the following procedure. The tubing was laid in a straight line and a 14.7 psi vacuum applied at one end. A cork with a thin thread attached was introduced at the other end and drawn through the tubing by the vacuum. A stronger thread was then attached to the fine thread and pulled through. Finally the cable was tied to the strong thread and it was pulled through. The plastic tubing with a swagelok connector screws into the top of the piezoblade. To prevent the cable from ever

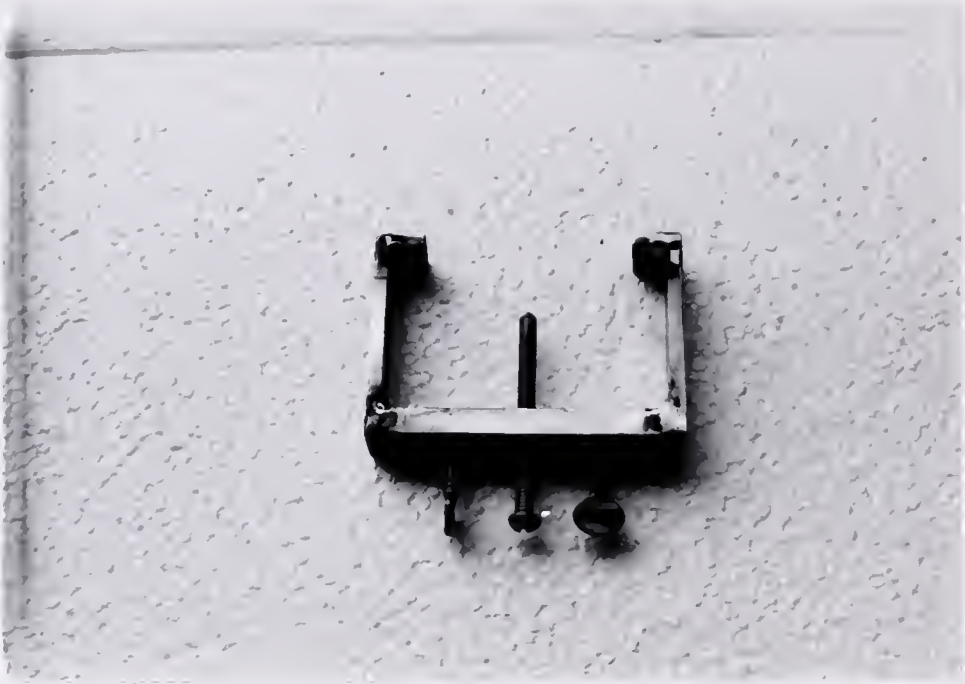


Figure 2-16. The De-airing Unit Holder

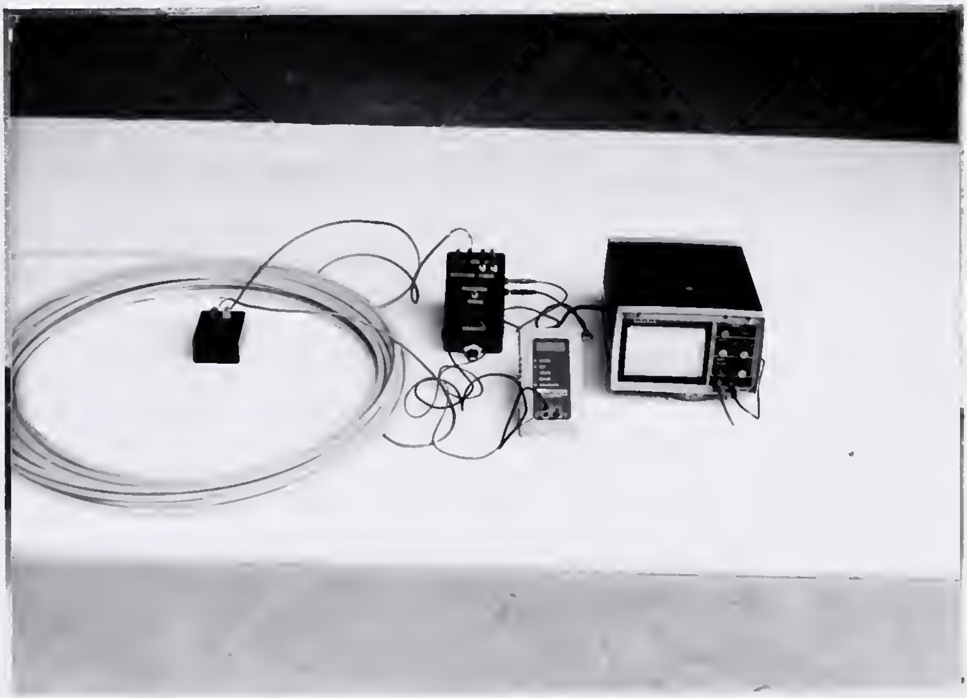


Figure 2-17. The Test Control and Recording Units

being accidentally pulled back up into the tubing, a simple twisted wire collar of large diameter was placed on the cable. In addition, the wires which connect the cable to the connector, Figure 2-12, are very thin and will break easily if any severe pulling on the cable should occur. These can be easily replaced.

2.5 Control and Recording Units

During the performance of a test, four electrical control/recording units are employed. These are shown hooked together ready for testing in the photograph of Figure 2-17. Because the piezoblade cable must be threaded through the string of drill rods it is not possible to have permanent electrical connectors or clips on the four wires. Instead, these exposed wires are attached to four alligator clamps in the Clamp Box, Figure 2-18. This then connects to the Control Box via banana jacks. The Control Box circuitry is shown in Figure 2-19. It has eight female banana jacks. Four connect via the Clamp Box to the transducer, two connect to a voltmeter and two to the recorder. A three position switch allows output of the excitation voltage or the signal voltage. The center switch position is off. Two 9-volt batteries provide the power while a rotary resistor switch, Figure 2-20, permits exact control of the excitation voltage.

The voltmeter used in the research was a B and K-Precision Model 2845 Autoranging Digital Multimeter

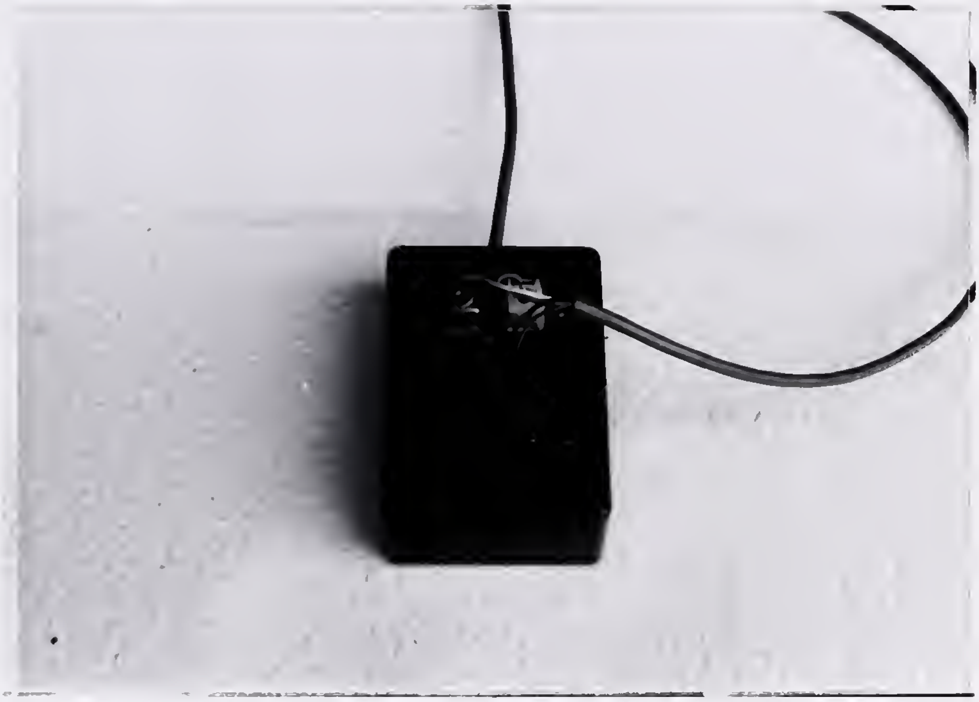
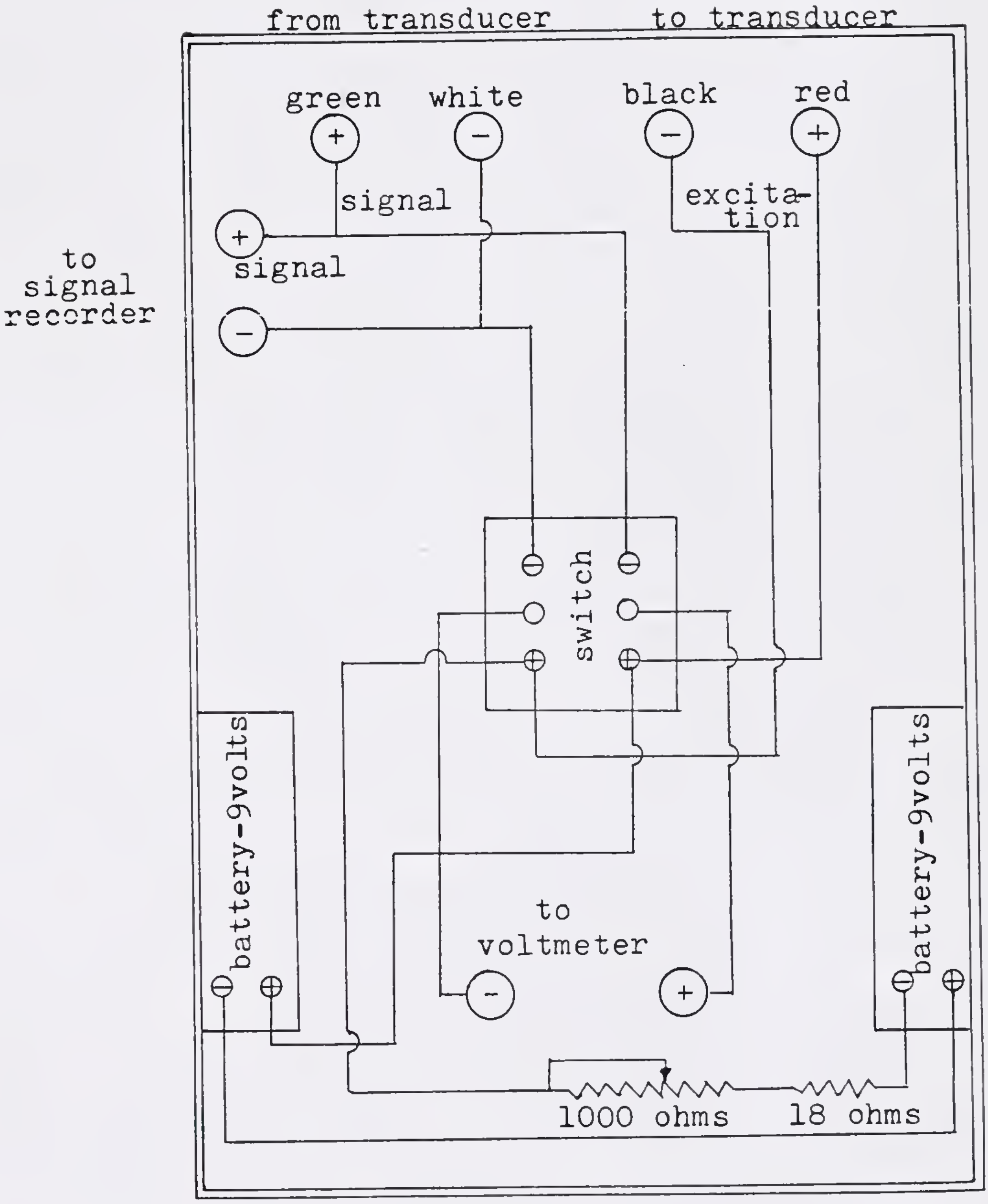


Figure 2-18. The Clamp Box



○ female banana jack

Figure 2-19. The Control Box Circuitry

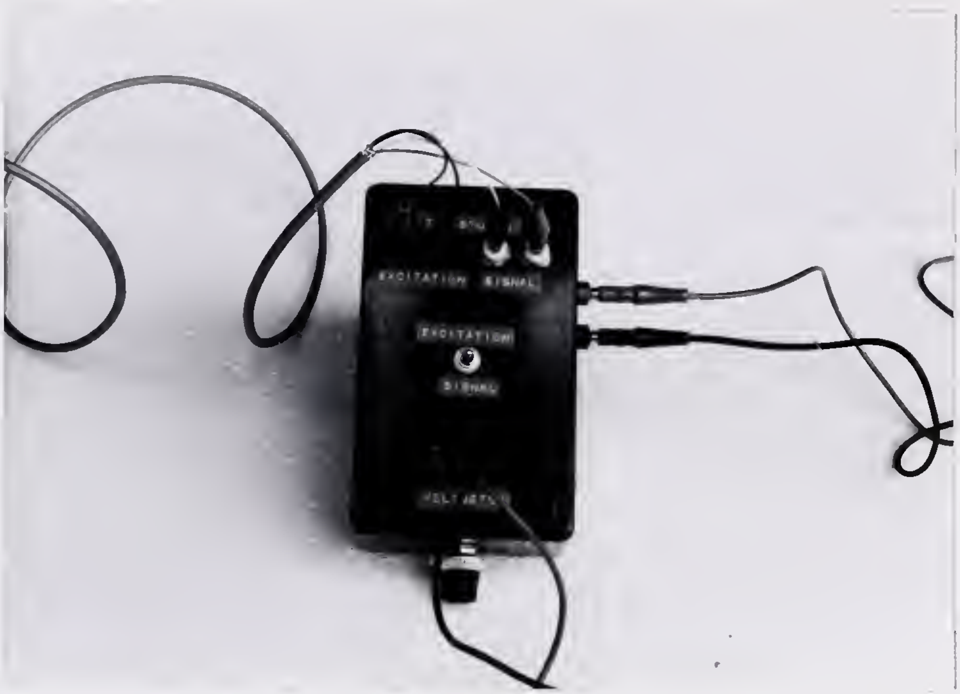


Figure 2-20. The Test Control Box

Figure 2-21. The Voltmeter

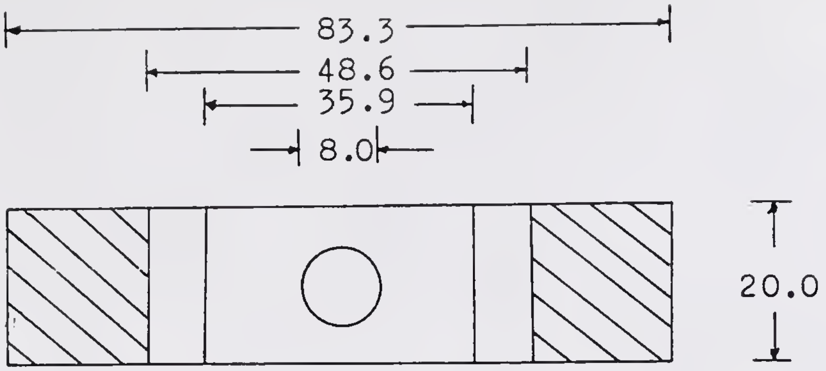
manufactured by Dynascan Corporation, Figure 2-21. This model is a high-accuracy instrument for measuring dc and ac voltage, dc and ac current, and resistance. Among the performance features are 0.1% accuracy, extensive overload protection and long battery life. Since the piezoblade penetrates the soil at a constant rate, a chart recorder was used to provide a permanent record of the pore pressure changes. The recorder used was manufactured by Linear Instrument Corporation, their model number 142. Figure 2-22. It is a multirange potentiometric, null balance servo recorder which can be powered either by mains electricity or by rechargeable batteries. The unit is small, weighs only 5 kilograms and is convenient for field testing. The chart width is 100 mm.

2.6 Calibration of the Pore Pressure Transducer

Before sealing the transducer permanently into the probe, it was necessary to calibrate it, i.e., experimentally establish the relationship between applied pressure in psi and the output signal in volts. To do this the special calibration unit shown in Figures 2-23 and 2-24 was built.

The device consists of a metal ring through which two holes were drilled and threaded. To the upper hole of Figure 2-23 was threaded plastic tubing connected to an accurate pressure regulator and gauge and to a gas source. Through the lower threaded hole was a screw by

Figure 2-22. The Linear Instrument Corporation Recorder



Section A-A
without holder and transducer

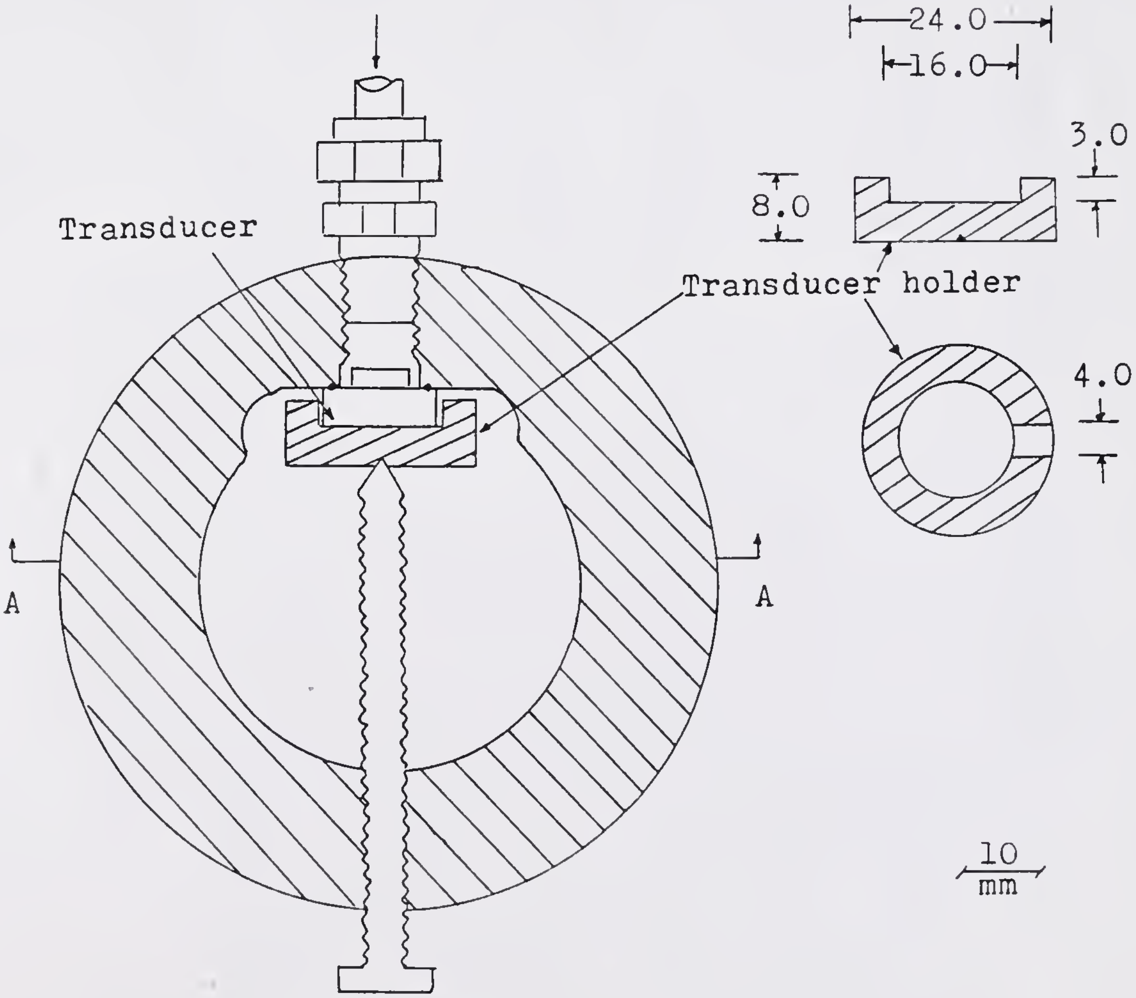


Figure 2-23. The Calibration Unit

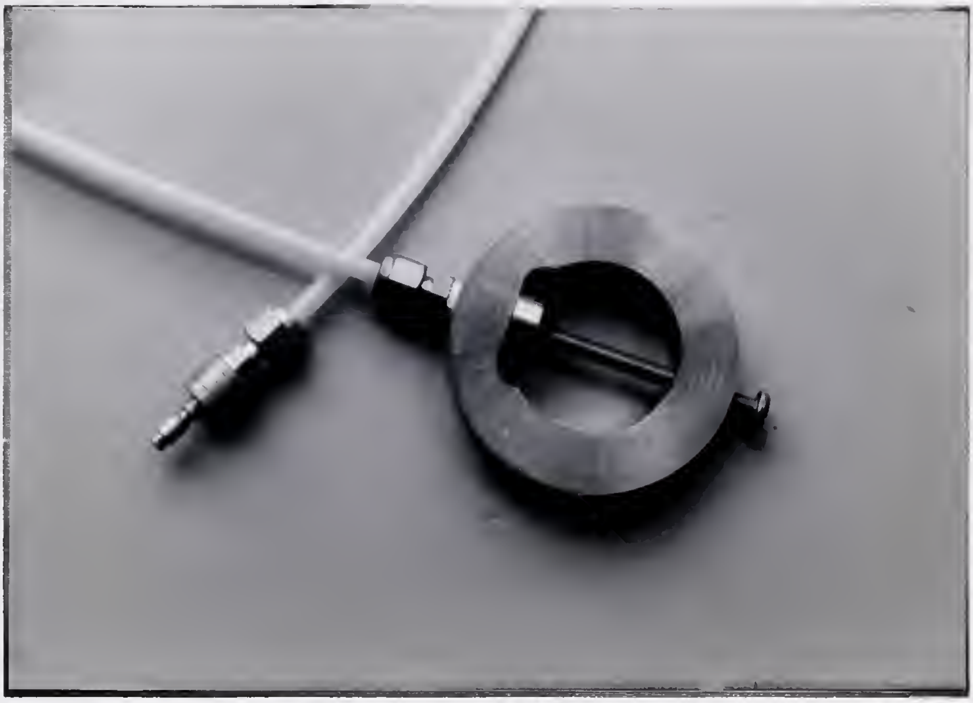


Figure 2-24. Photograph of the Calibration Unit

which the pressure transducer and its holder were tightened against an O-ring, forming the pressure cavity. Known increments of gas pressure were applied and the transducer output recorded. The excitation voltages were also varied during the test, between 14.3 and 16.1 volts in 0.1 volt increments. Figure 2-25 shows the final calibration curves. The calibrations were all linear over the pressure range tested, with a slope for the 15 volt excitation giving a relationship of 3.54 mv/psi. The effect of varying the excitation voltage on the output voltage was to increase it by 1 to 2 mv for each 0.1 volt increase in input voltage. In a particular output signal the effect of a 0.1 volt change in excitation voltage is thus a 0.28 to 0.56 psi change in pressure. These curves can be used to interpolate the results of a test if the input voltage is not exactly 15.0.

2.7 De-airing the Piezoprobe

A very critical part of any laboratory or field testing method which attempts to measure pore water pressure is the de-airing of the unit. Test results cannot be relied upon with confidence if there has been failure to remove all gas bubbles from the system, or if a fluid has been used from which gas may come out of solution during testing, Baligh et al. (1980).

In this research, a special piezoblade de-airing unit was constructed. It consists of four parts, the holder Figure 2-26, the cylinder Figure 2-27, the top cap section

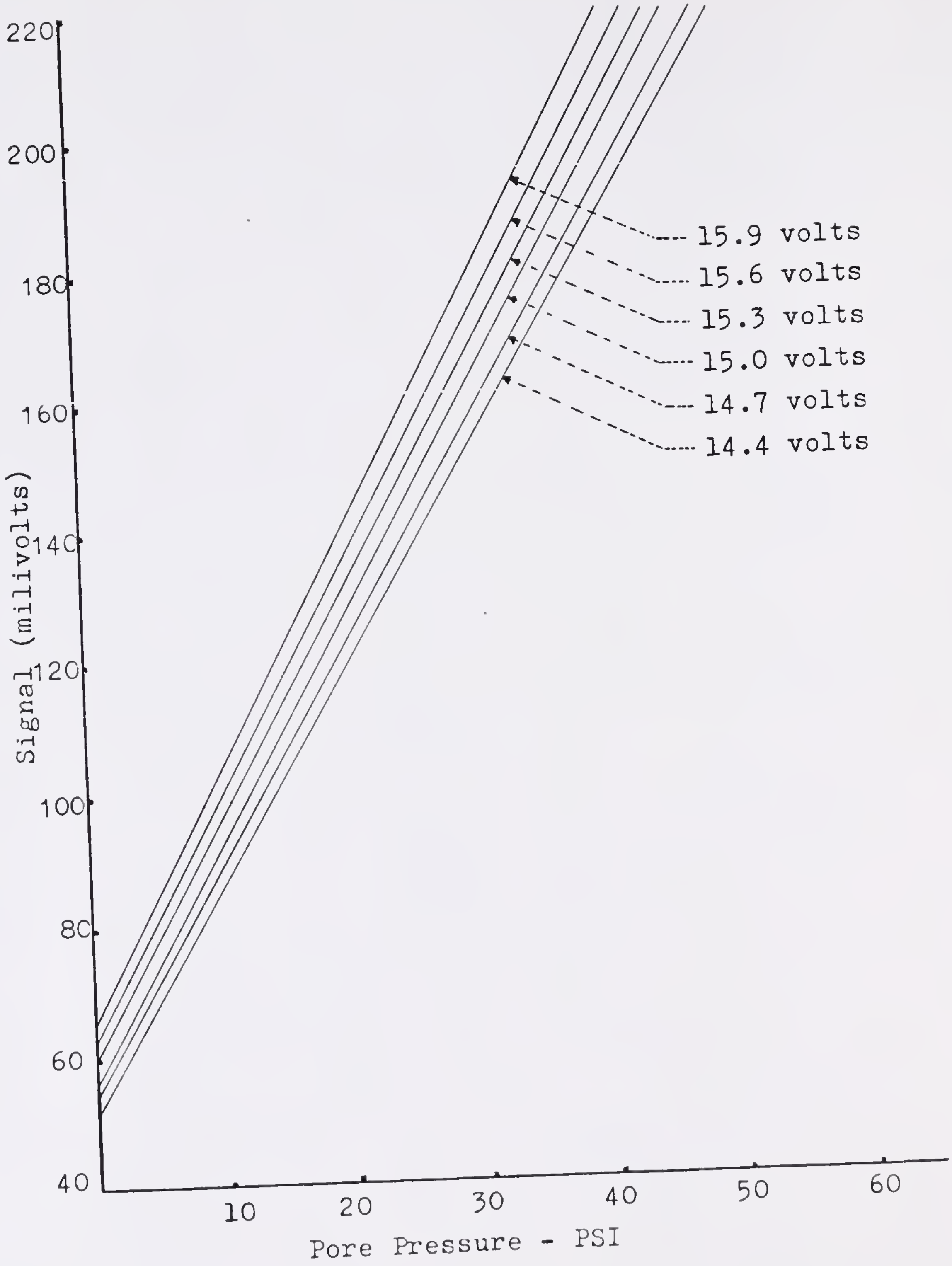


Figure 2-25. Calibration Curves for Different Excitation Voltages

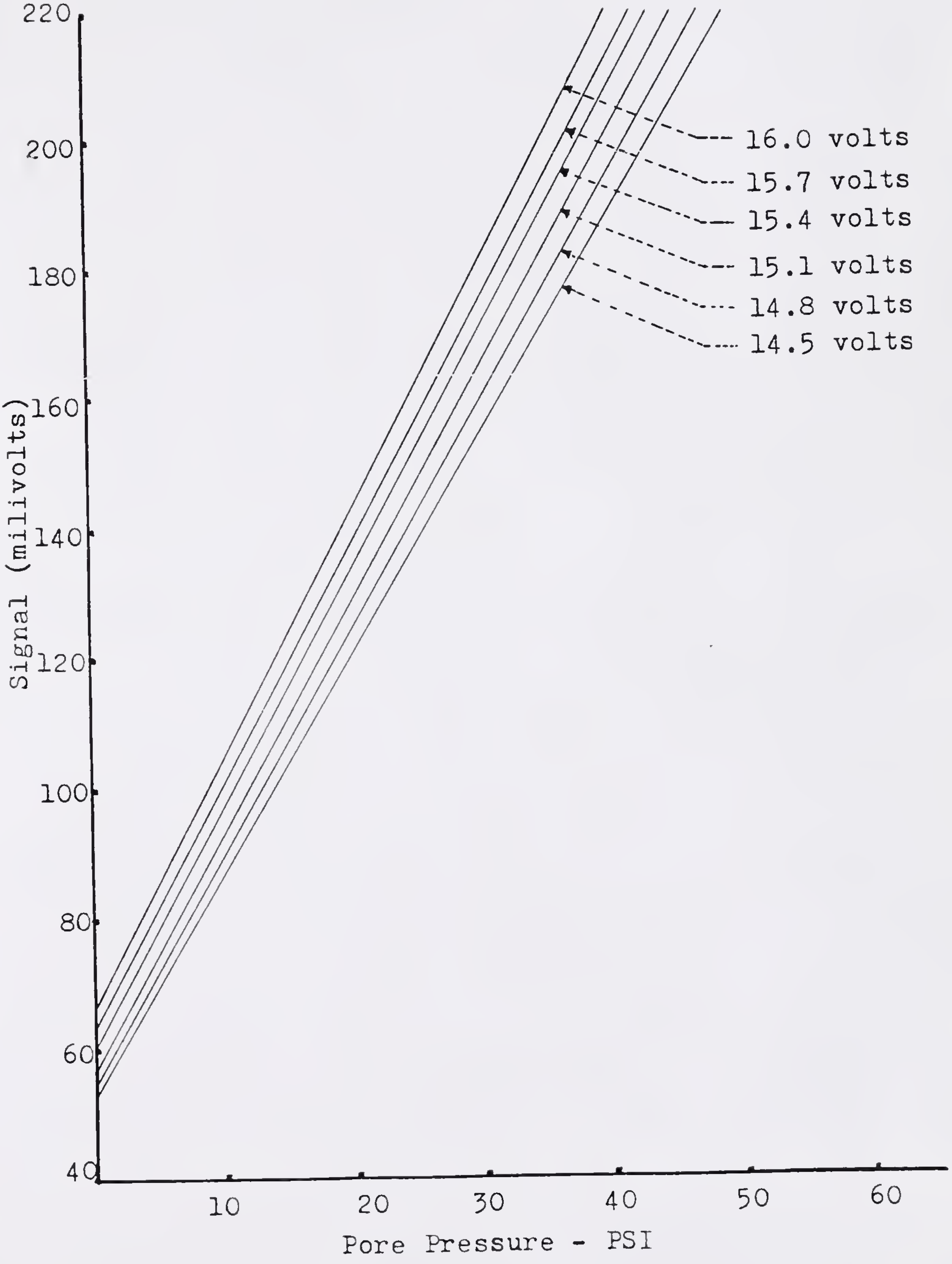


Figure 2-25. (Continued)

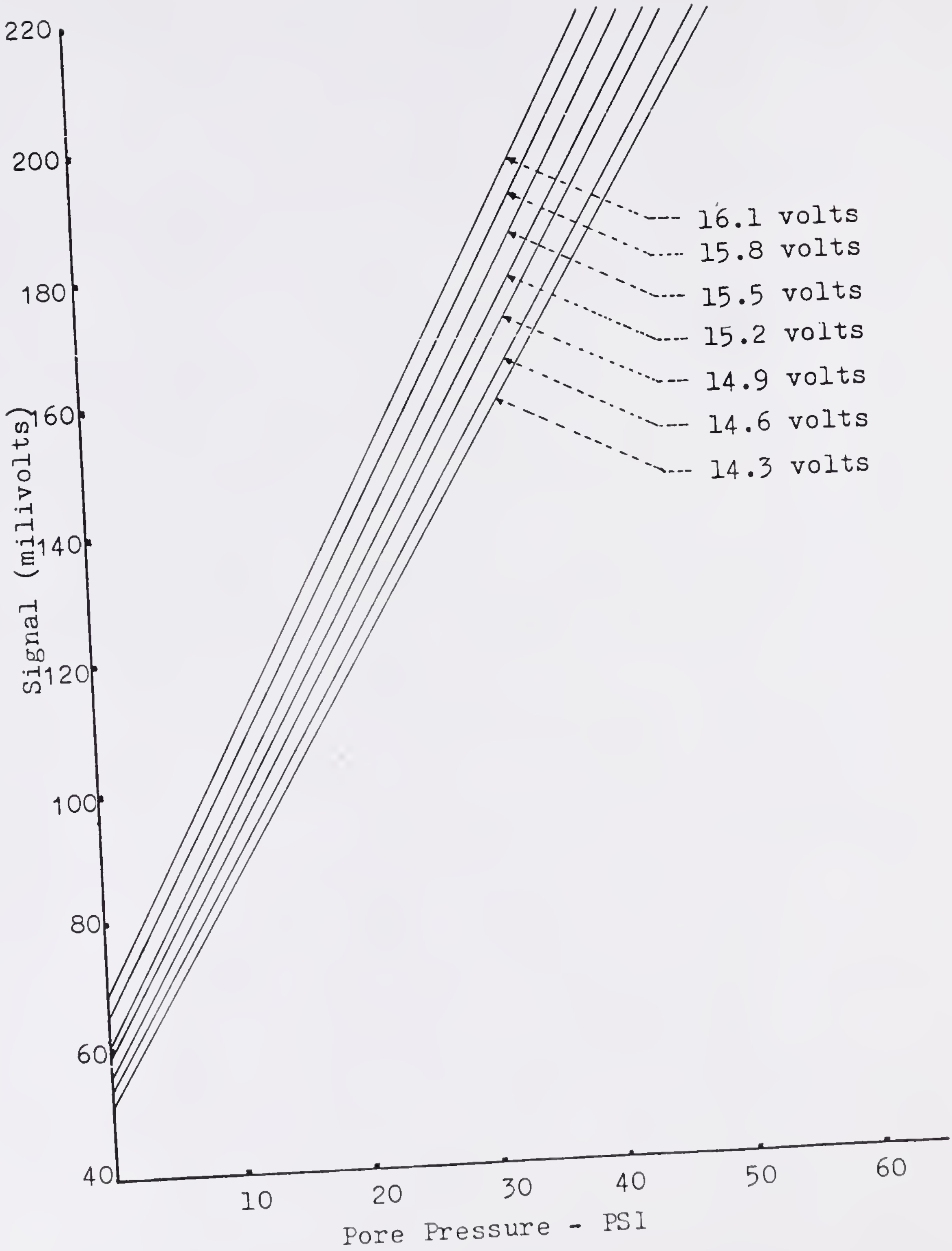


Figure 2-25. (Continued)

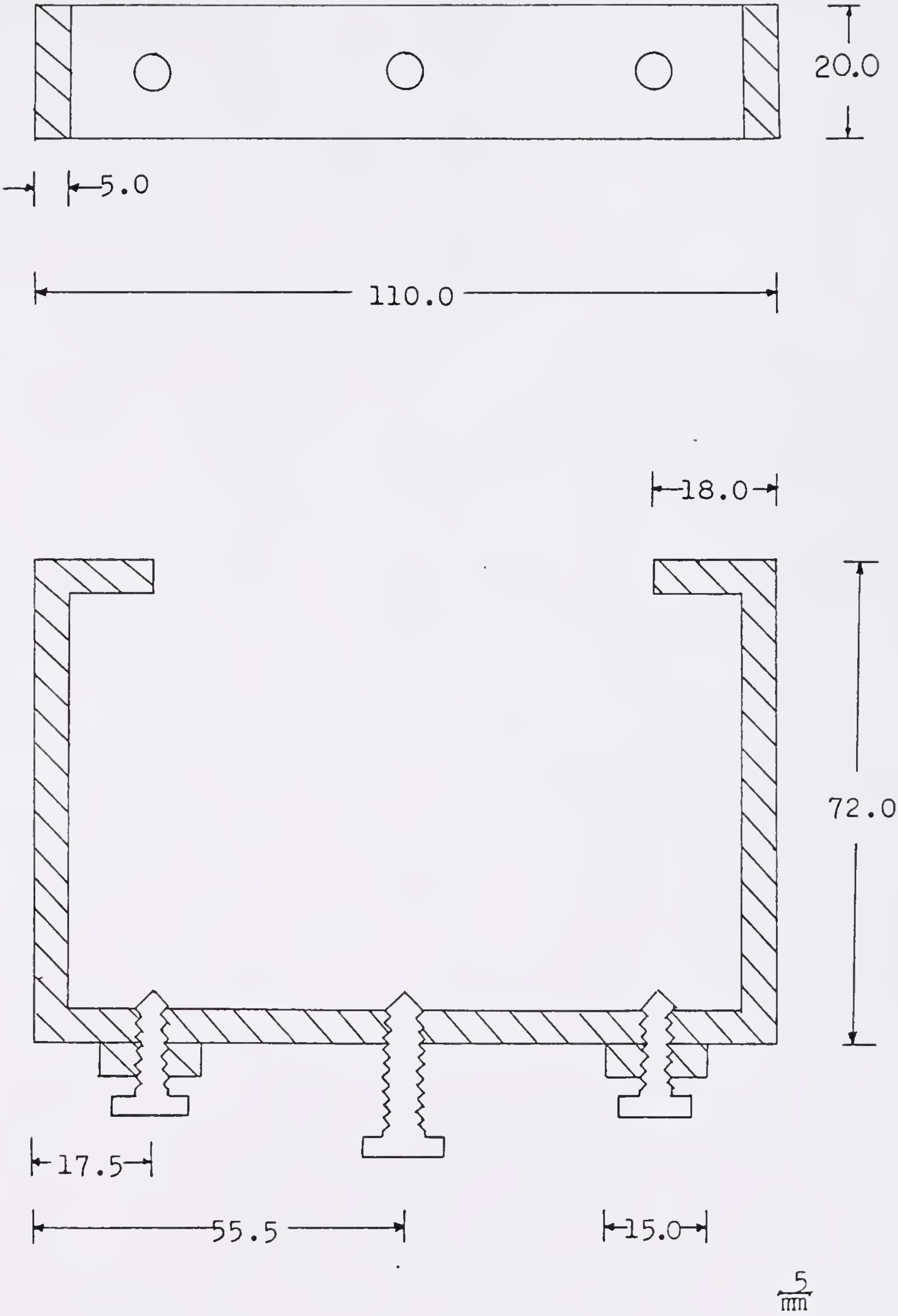


Figure 2-26. The De-airing Unit Holder

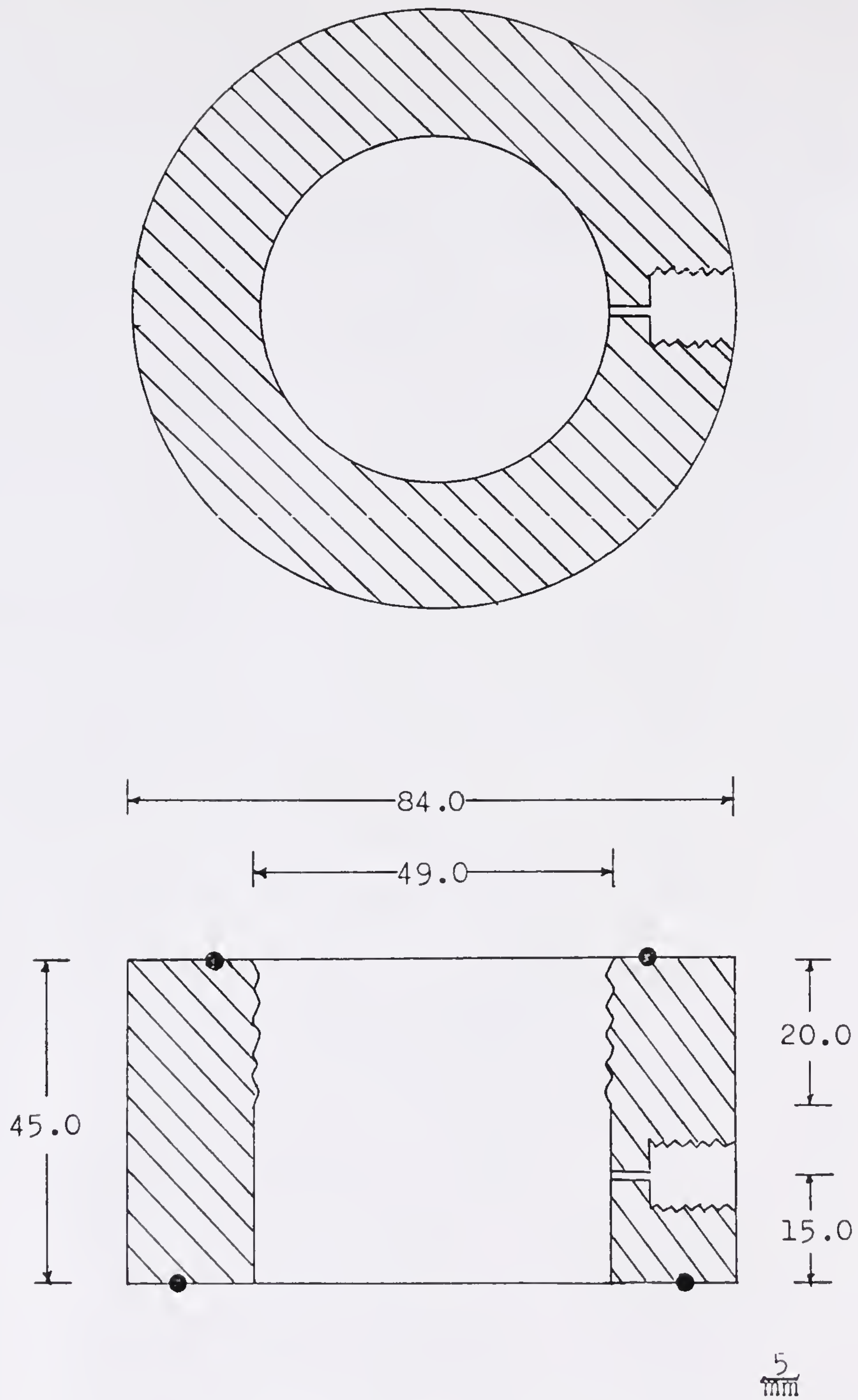


Figure 2-27. The De-airing Unit Cylinder

Figure 2-28, and the inside screen Figure 2-29. The assembled unit is shown in Figure 2-30. The thick-walled, stainless steel cylinder with O-rings top and bottom is placed on the piezoblade and tightened down by means of the two outer screws on the holder. The center screw is used only during the cementing in place of the transducer holder, described in Section 2.3. The porous stone and its holder are placed in the cylinder and the screen put in place. About a one centimeter depth of de-aired water is added and the top cap screwed on.

The unit is then turned upside-down, such that the water is now sitting on the lucite top and the porous stone and holder are on the screen. A vacuum is applied via the cylinder's quick connect and all air removed from the space above the water, from the piezoblade cavity and from the porous stone. The unit is then turned the right way up and the water allowed to flood the piezoblade cavity. The vacuum is disconnected and air allowed in on top of the water. The top cap can then be unscrewed and the screen removed. The porous stone and holder are assembled and screwed in place, underwater.

The piezoblade and the de-airing assembly are then placed in a bucket of de-aired water and the de-airing unit unscrewed. Finally the de-aired and saturated piezoblade is placed in a plastic bag full of water and sealed. It is ready for field testing. Figures 2-31 and 2-32 show photographs of the de-airing unit.

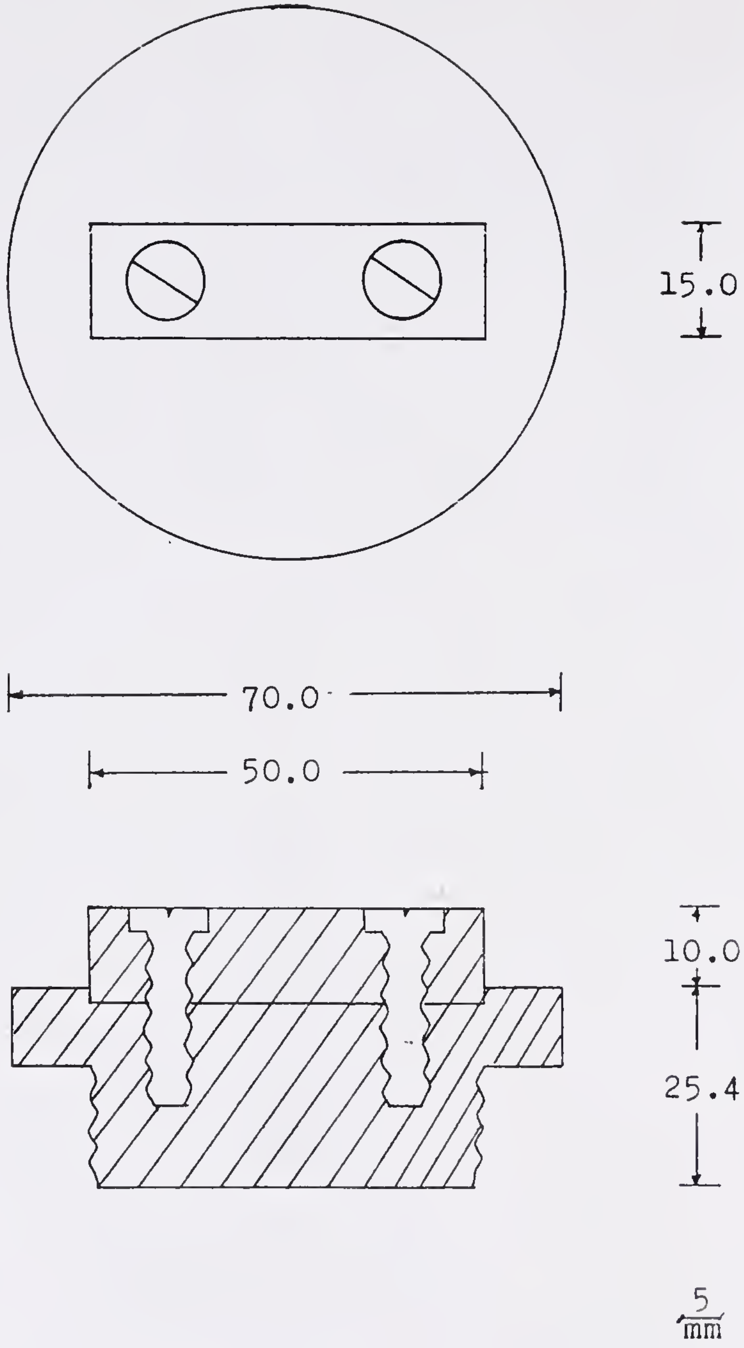


Figure 2-28. The De-airing Unit Top Section

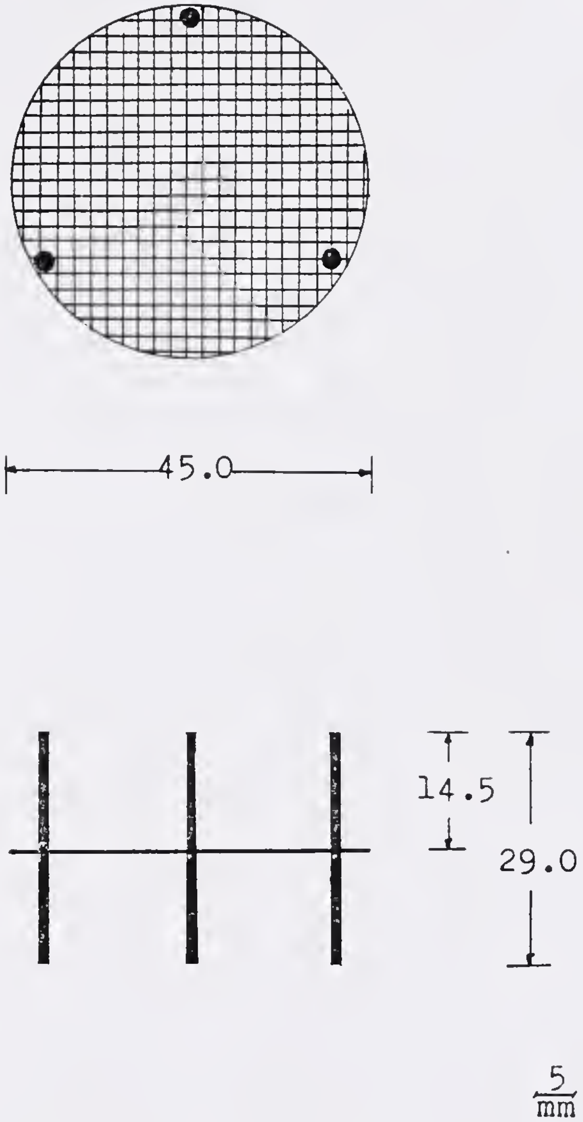


Figure 2-29. The De-airing Unit Screen

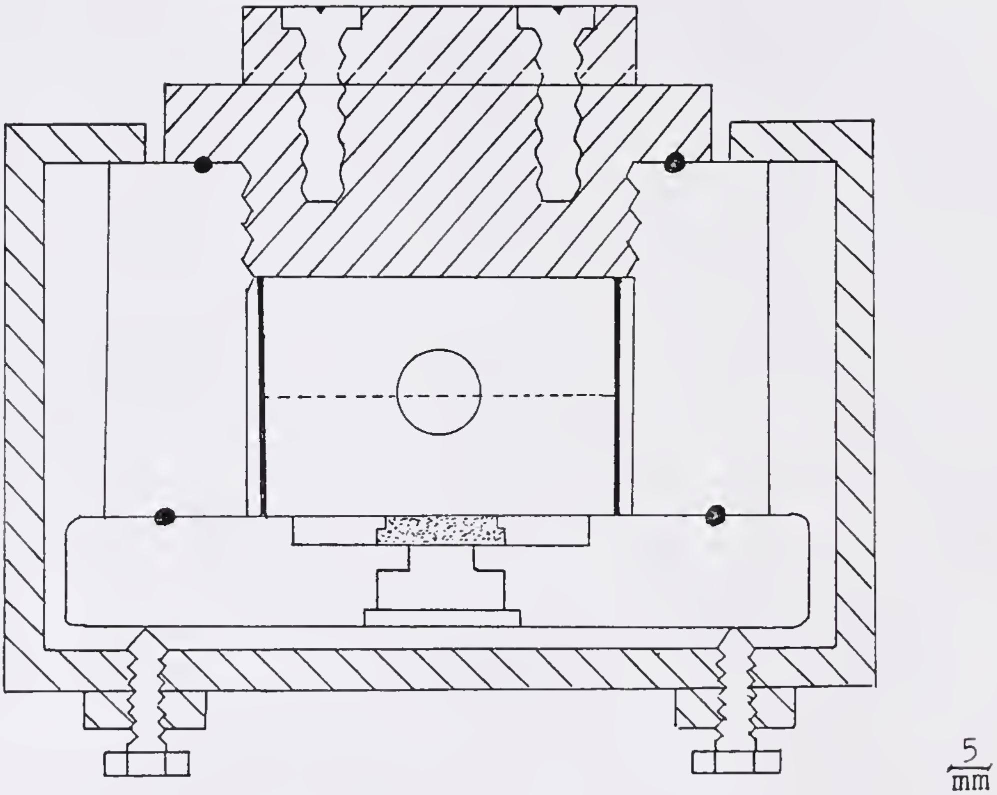
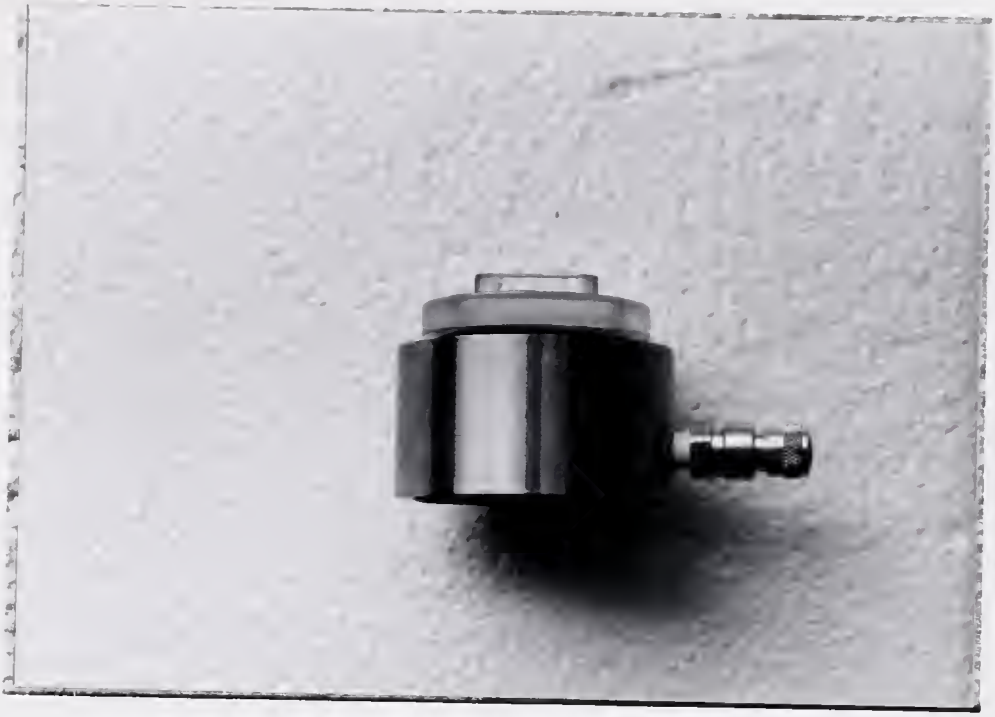


Figure 2-30. The Assembled De-airing Unit



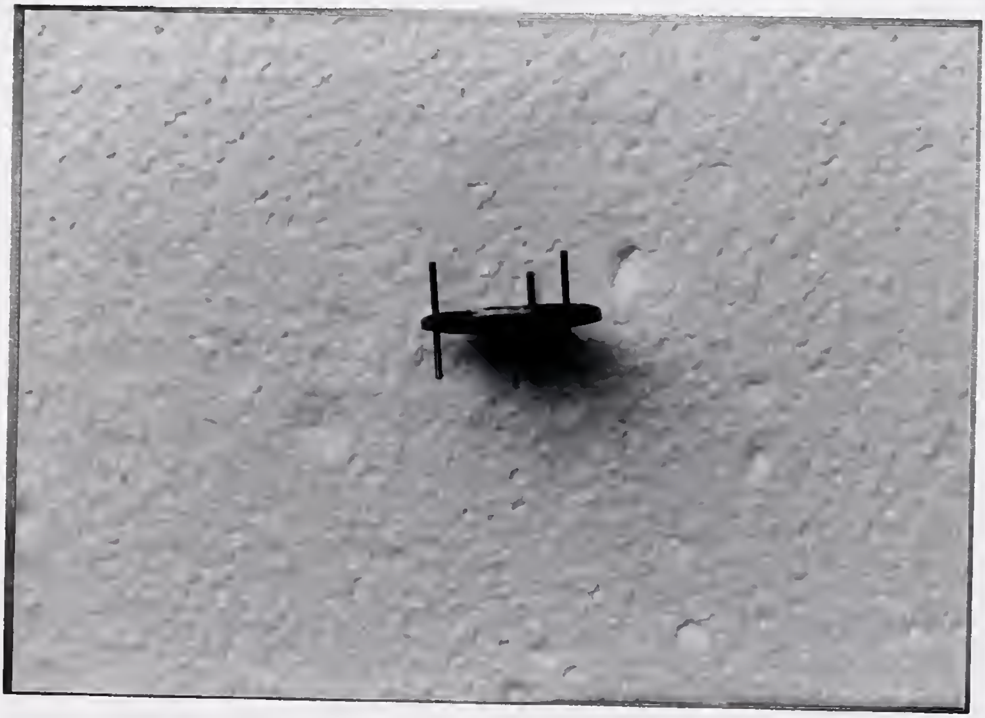
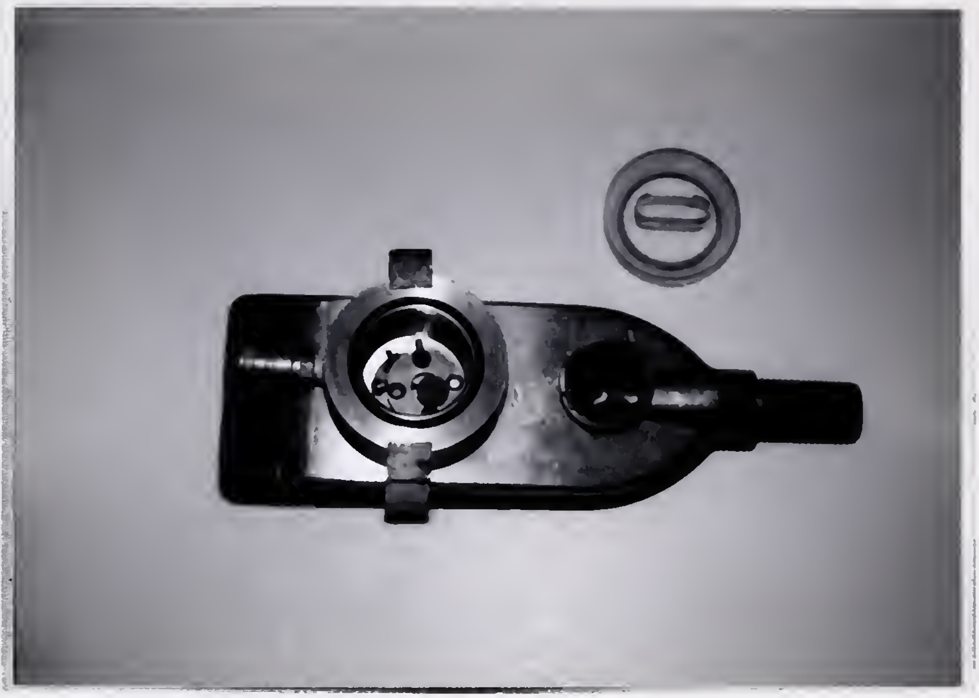


Figure 2-31. Photographs of the De-airing Unit's Cylinder, Top and Screen



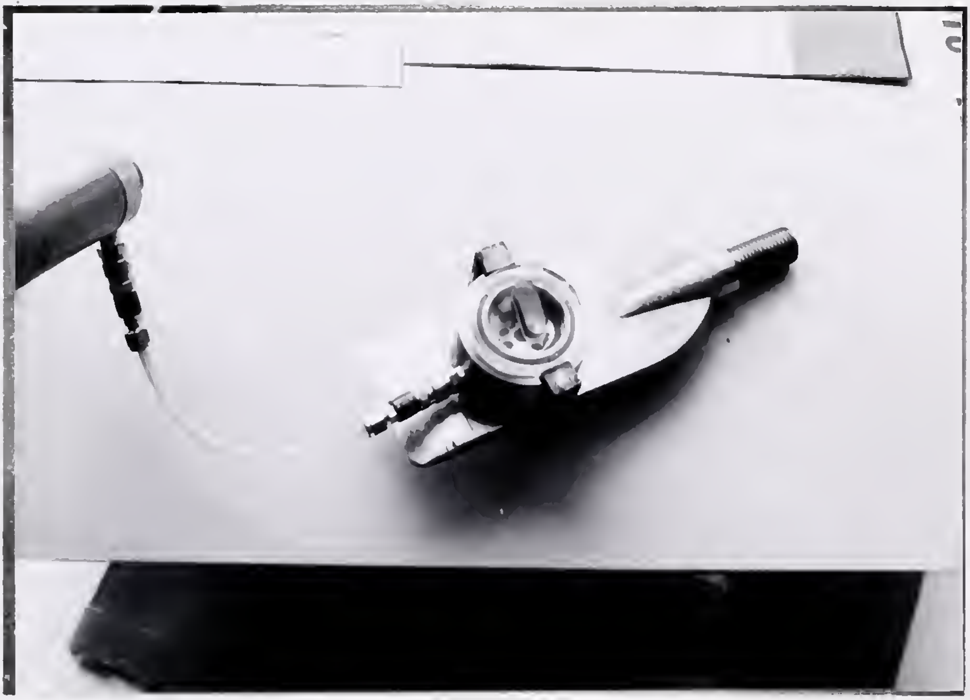


Figure 2-32. Photographs of the De-airing Unit and the Piezoblade

CHAPTER 3

OTHER INSITU TEST DEVICES

3.1 Introduction

In the field testing program associated with this research a large number of dilatometer and cone penetration tests were performed adjacent to the piezoblade soundings. This chapter very briefly describes these devices, the test procedures and the relevant data reduction methods.

3.2 The Dilatometer Test

The flat plate dilatometer was developed in Italy by Dr. Silvano Marchetti, professor of soil mechanics at L'Aquila University, Marchetti (1975, 1980). The device was originally designed to measure the insitu soil modulus for laterally loaded piles. However, extensive field testing has generated a number of correlations which now permit the evaluation of many other geotechnical parameters.

3.2.1 The Equipment

The equipment consists of three major components, the blade, the control unit, and the connecting cable. Figure 3-1 is a photograph of all the test equipment and includes,

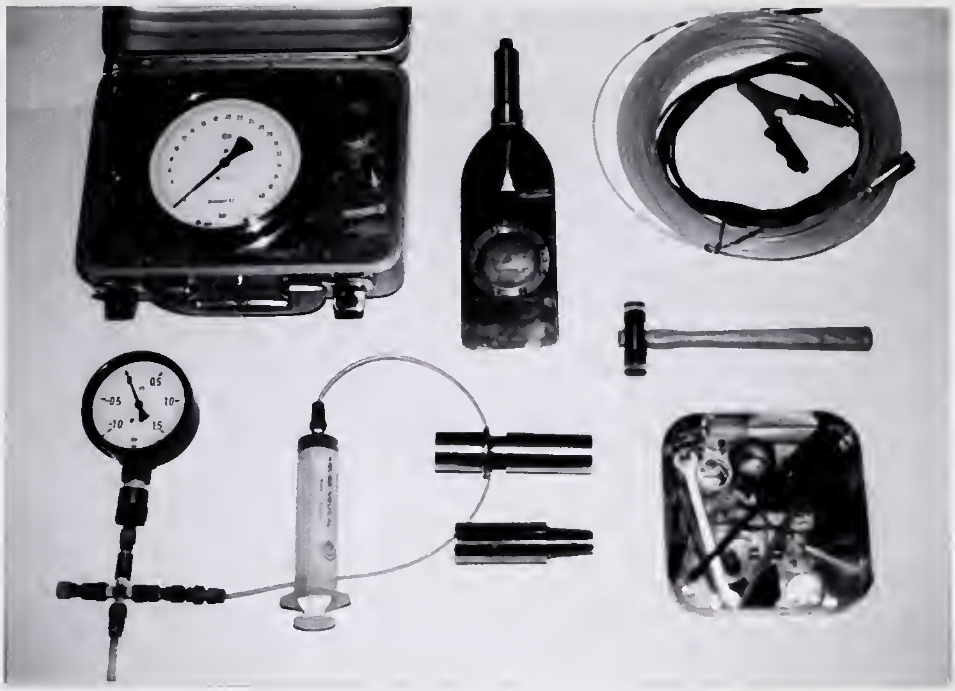


Figure 3-1. The Dilatometer Test Equipment

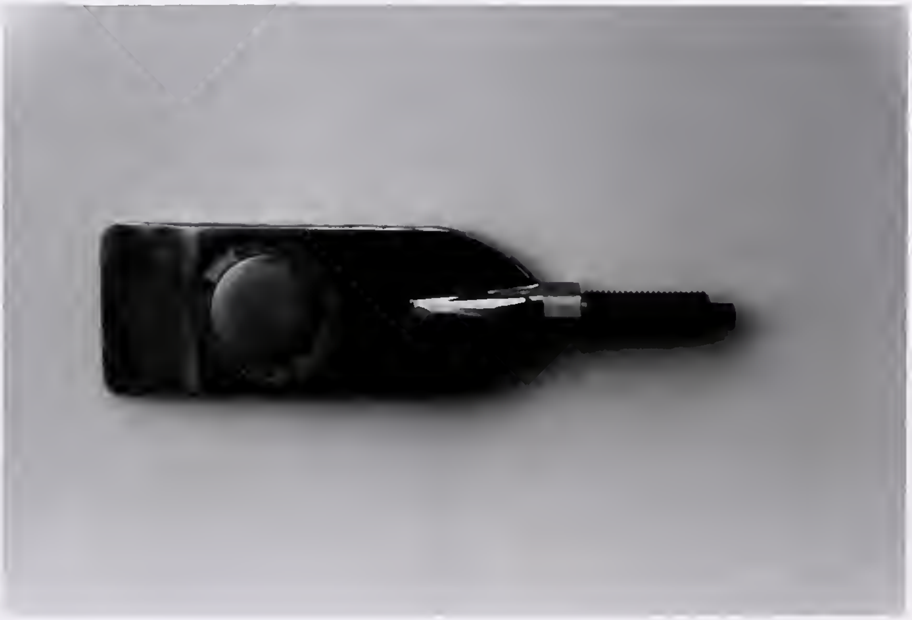


Figure 3-2. The Flat-blade Dilatometer

besides the three basic units, some calibration equipment and a box of tools. The blade is shown above in Figure 3-2. It is a stainless steel plate with a sharpened bottom edge and is threaded for connection to a string of drill rods. On one face of the blade is a thin circular expandable, steel membrane. The test consists of pushing the plate vertically into the ground and stopping at frequent intervals to determine the gas pressure behind the membrane needed to expand it 1 mm into the soil. The cable which connects the dilatometer to the control unit is threaded through the drill rods and is a combined gas and electrical line. The control unit, Figure 3-3, consists of a pressure gauge and regulator by which gas pressure is sent to the dilatometer, and a galvanometer and buzzer which indicate when the desired displacement has been achieved.

3.2.2 Dilatometer Test Procedure

Prior to testing, a brief calibration is required. The syringe and pressure/vacuum gauge of Figure 3-4 are attached to the pressure control unit instead of the normal gas source. A vacuum, applied by drawing the syringe plunger, pulls the dilatometer membrane into contact with its seating, completing an electric current and activating the buzzer. This vacuum pressure, ΔA, is recorded. A pressure is then applied by pushing in the plunger and the pressure ΔB noted when the buzzer is

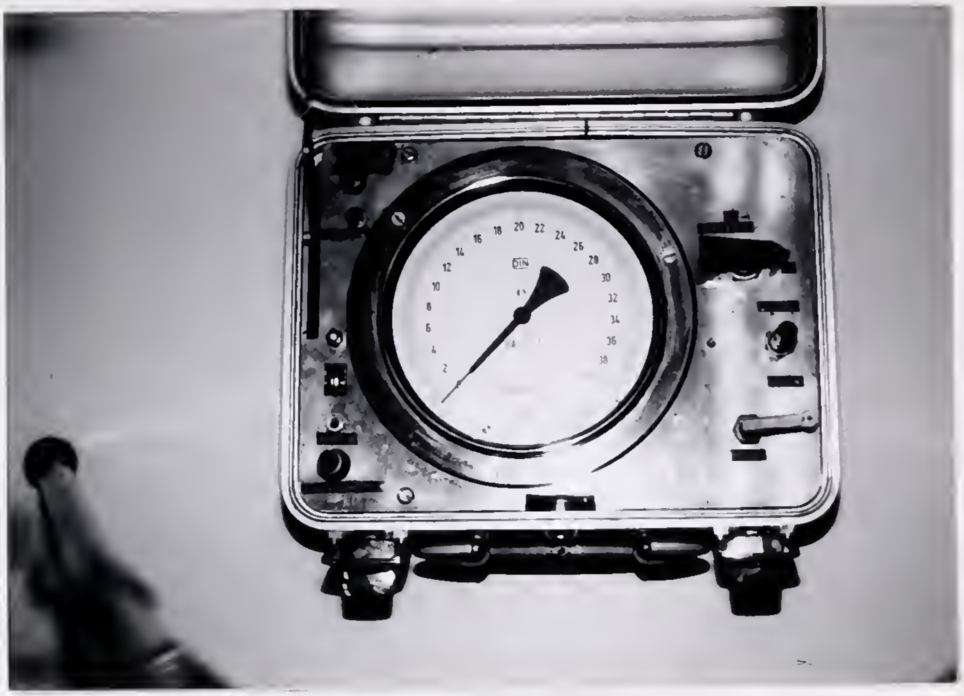


Figure 3-3. The Control Unit

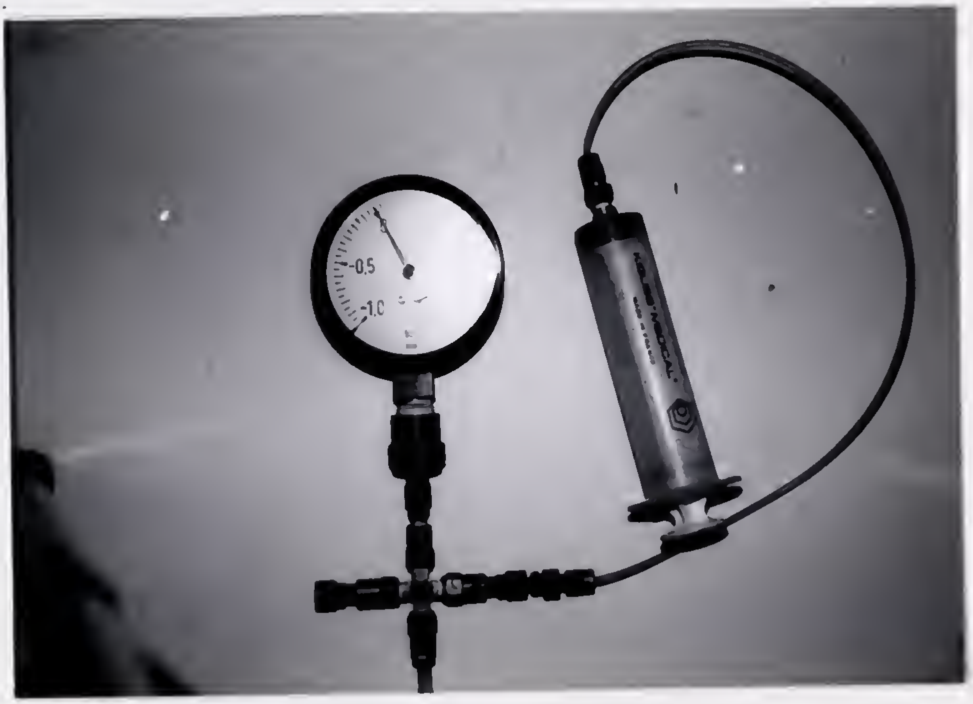


Figure 3-4. The Calibration Equipment

reactivated indicating a membrane expansion of 1 mm. These procedures are repeated until consistent values are obtained.

For the field test, the dilatometer is pushed into the ground at a slow constant rate by the University of Florida cone penetration test truck. At 20 cm depth intervals, penetration is ceased and a test performed. During insertion the soil pressure on the membrane presses it onto its seating and the buzzer sounds. To run the test, the gas pressure behind the membrane is slowly increased by means of the central panel regulator. When the membrane separates from its seating the buzzer deactivates, and the pressure acting, A, is recorded. When the buzzer reactivates, the 1 mm expansion has been achieved. This pressure, B, is also recorded and the equipment immediately vented so as not to overstress the metal membrane. The dilatometer is then ready to be penetrated to the next test depth.

3.2.3 Dilatometer Data Reduction

The dilatometer parameters I_D, K_D, and E_D are calculated by means of the following equations, Marchetti (1980),

$$P_0 = 1.05(A-Z_M+\Delta A)-0.05(B-Z_M-\Delta B) \tag{3-1}$$

$$P_1 = B-Z_M-\Delta B \tag{3-2}$$

$$\Delta P = P_1-P_0 \tag{3-3}$$

$$I_D = (P_1 - P_0)/(P_0 - U_0) \qquad (3\text{-}4)$$

$$K_D = (P_0 - U_0)/\bar{\sigma}_v \qquad (3\text{-}5)$$

$$E_D = 38(P_1 - P_0) \qquad (3\text{-}6)$$

where

I_D = Material index

K_D = Horizontal stress index

E_D = Dilatometer modulus

P_0 = Corrected first reading. Pressure to separate the membrane from its seating

P_1 = Corrected second reading. Pressure to expand the membrane center one millimeter into the soil.

U_0 = Hydrostatic pore water pressure

A = First dilatometer reading

B = Second dilatometer reading

ΔA = Free-air correction for A

ΔB = Free-air correction for B

Z_M = Zero pressure gauge reading when control unit is vented

$\bar{\sigma}_v$ = Vertical effective stress at depth of membrane center

P_0 and P_1, the corrected values of A and B, depend on the calibration pressures and on the geometry of the measuring device. The equations are derived in Marchetti's (1980) paper.

The Material Index I_D correlates well with particle size, ($I_D < 0.6$ = clay, $0.6 \leq I_D \leq 1.8$ = silt, $I_D > 1.8$ = sand), Figure 3-5.

The Horizontal Stress Index K_D is of the form of a lateral stress ratio, i.e., the ratio of an effective lateral pressure to the vertical effective stress.

The Dilatometer Modulus E_D was developed from a theory of elasticity solution to the following problem: a load on the surface of a half-space with no normal displacement of the surface permitted outside the loading.

$$S = \frac{2D\ P}{\pi} \cdot \frac{(1-\nu^2)}{E} \tag{3-7}$$

where

E = Young's modulus

ν = Poisson's ratio

D = Membrane diameter = 60 mm

S = Movement of membrane = 1 mm

therefore,
$$1 = \frac{2(60)\ P}{\pi} \cdot \frac{(1-\nu^2)}{E}$$

$$E_D = \frac{E}{1-\nu^2} = 38\Delta P \tag{3-8}$$

Experimental correlation with the dilatometer parameters defined above allows the evaluation of several geotechnical parameters, including K_0, OCR, $M(=1/m_v)$, C_u and ϕ. Details of all the correlations are given in the Marchetti (1980) paper. Those for the overconsolidation ratio (OCR) and the constrained modulus (M), which are used in Chapter 7 of this thesis, are listed below.

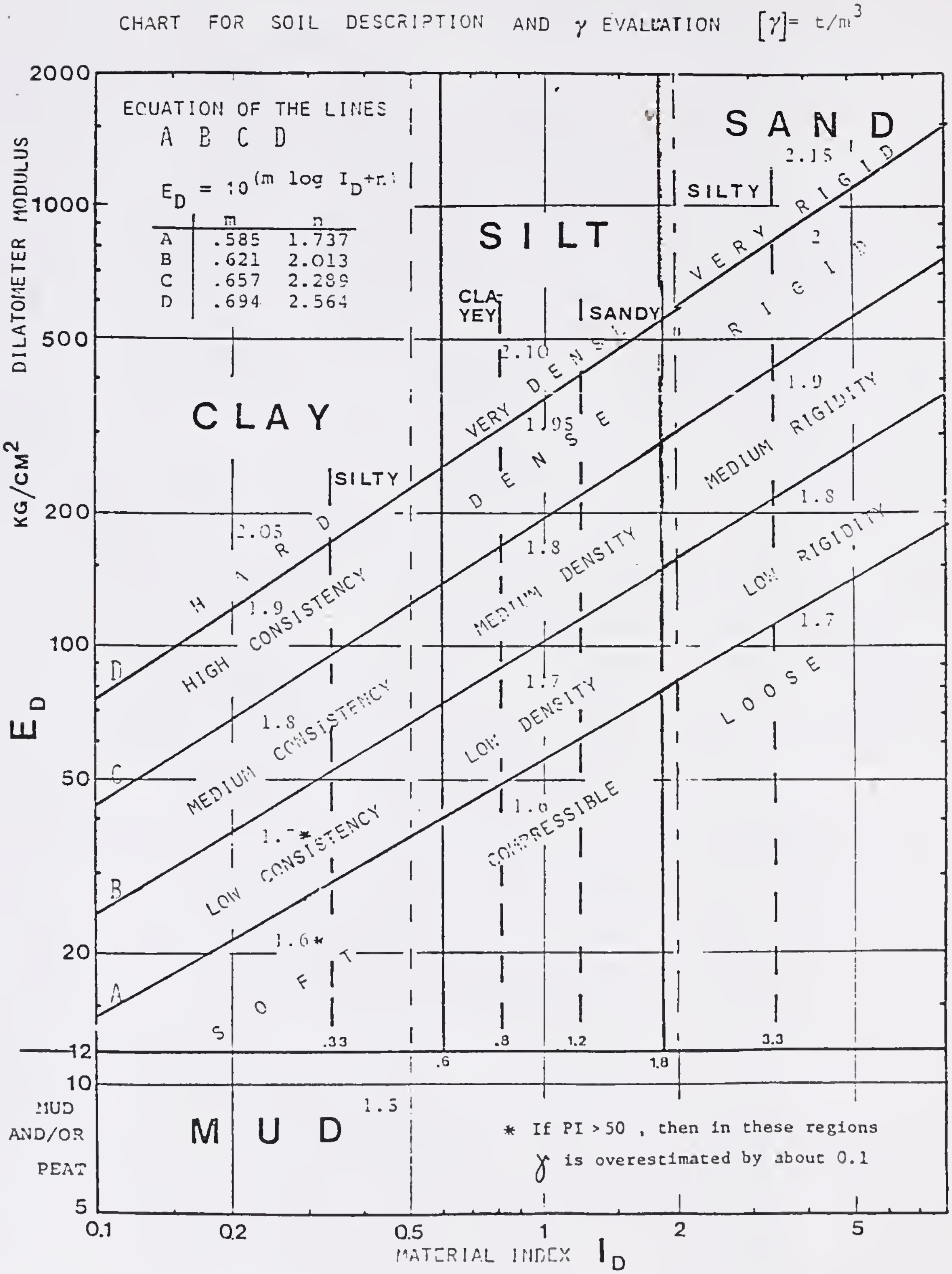


Figure 3-5. Dilatometer Soil and Density Evaluation Chart (Marchetti and Crapps, 1981)

$$\text{If } I_D<1.2 \qquad OCR = (.5K_D)^{1.56} \qquad (3\text{-}9)$$

$$\text{If } I_D>2.0 \qquad OCR = (.65K_D)^{1.91} \qquad (3\text{-}10)$$

$$\text{If } 1.2<I_D<2.0 \qquad OCR = (mK_D)^{n} \qquad (3\text{-}11)$$

where

$$P = (I_D-1.2)/0.8 \qquad (3\text{-}12)$$

$$m = 0.5 + 0.17P \qquad (3\text{-}13)$$

$$n = 1.56 + 0.35P \qquad (3\text{-}14)$$

If the value of OCR predicted from the dilatometer test is less than 0.8, it is written as $OCR<0.8$, because it is not within the range of valid correlation.

$$M=R_M \cdot E_D \qquad (3\text{-}15)$$

where

$$\text{If } I_D\leq 0.6 \qquad R_M = 0.14 + 2.36 \log K_D \qquad (3\text{-}16)$$

$$\text{If } I_D\geq 3.0 \qquad R_M = 0.5 + 2.0 \log K_D \qquad (3\text{-}17)$$

$$\text{If } 0.6<I_D<3.0 \qquad R = R_{m,o} + (2.5 - R_{m,o})\log K_D \qquad (3\text{-}18)$$

$$R_{m,o} = 0.14 + 0.15(I_D-0.6) \qquad (3\text{-}19)$$

If the calculated value of R_M is less than 0.85, then 0.85 is used in the determination of M.

3.3 The Cone Penetration Test

The cone penetration test or Dutch cone test originated in Holland and consists of pushing a solid tip penetrometer into the ground at a constant rate. Surface measurements of the pressure required for penetration are made. Theoretical analyses and experimental correlations permit an evaluation of soil type and many geotechnical parameters.

3.3.1 The Equipment and Test Procedure

In this research a Begemann friction tip was used, Figure 3-6. This 60°, $10\ \mathrm{cm}^2$, mechanical tip requires the use of an outer and inner loading rod system. When a load is applied to the outer rod, the tip, in its collapsed state, Figure 3-6(a), is penetrated to the desired test depth. A load is then applied to the inner rod, which first penetrates only the point, Figure 3-6(b). The measured pressure permits the calculation of q_c, the cone end bearing. Further penetration causes the $150\ \mathrm{cm}^2$ friction sleeve to be engaged and both the point and the sleeve move, Figure 3-6(c). From this pressure is calculated $q_c + f_s$, where f_s is the unit side friction. Finally, the tip is again collapsed and penetrated to the next test depth by transferring the load to the outer rods.

The University of Florida cone truck was used to penetrate the Begemann tip. The truck, which weighs more than 13 tons, provides the necessary reaction while the

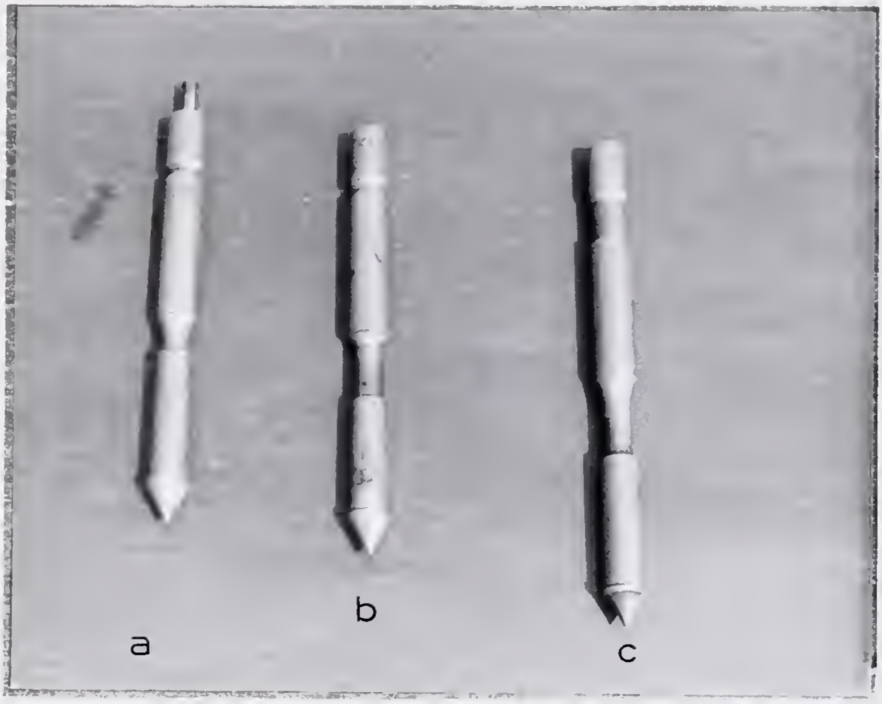


Figure 3-6. Begemann Tips

tip is penetrated by means of a hydraulic ram. Pressures are read from two Bourdon gauges, a low range 0 - 100 kg/cm^2 and a high range 0 - 600 kg/cm^2.

Depths and gauge pressures are usually punched into a programable hand calculator or main line computer and the data automatically reduced to give q_c, f_s and FR, the friction ratio. Plots of q_c and FR versus depth are available from the computer, see for example Appendix C.

3.3.2 Cone Penetration Test Correlations

A large number of primarily experimental correlations are available for determining geotechnical parameters and for foundation design. Schmertmann (1977a) contains detailed descriptions of many such correlations, including evaluation of soil type (Figure 3-7 and Table 3-1) relative density, sensitivity, OCR, C_u, ϕ', and design procedures for piles, settlement of footing on sand, consolidation settlement of clays and compaction control.

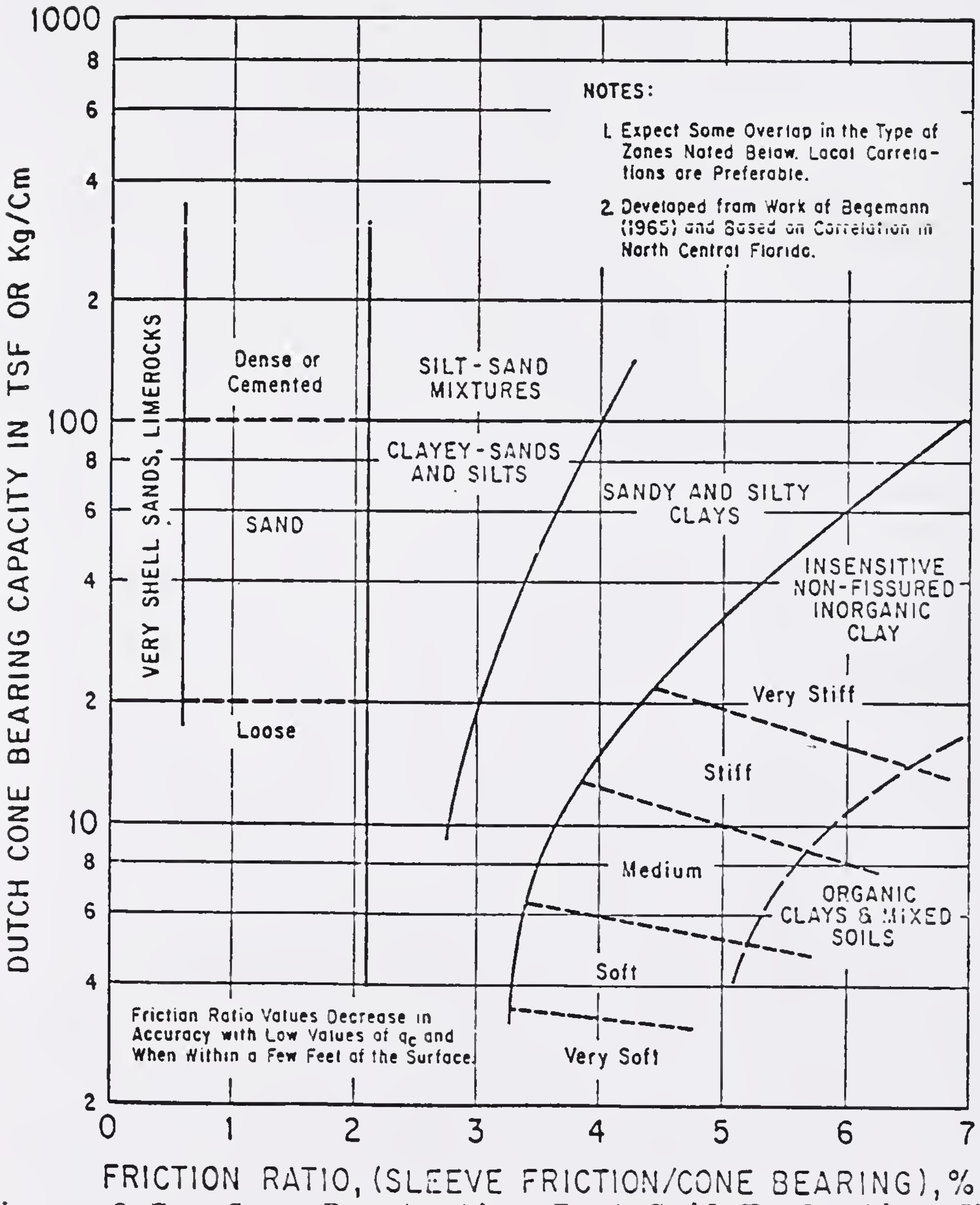


Figure 3-7. Cone Penetration Test Soil Evaluation Chart (Schmertmann, 1977b)

Table 3-1. Values of f_s/q_c for Loose Medium and Dense Sands (Schmertmann, 1977b)

Sand (D_r)	Loose 30%	Medium 60%	Dense 90%
K_p	1	2	4
tan δ (Steel)	1/3	1/3	1/3
q_c (bar)	20	80	220
f_s (bar)	1/3	2/3	4/3
f_s/q_c	1/60	1/120	2/330

K_p = Passive stress ratio

f_s/q_c = FR = Friction ratio

CHAPTER 4

FIELD TESTING

4.1 Introduction

The purpose of the field testing was to test the newly developed piezoblade and to establish relationships between it and the dilatometer. Of primary importance was an evaluation of the drainage conditions around the dilatometer in different soils. This was determined by using the identically shaped piezoblade adjacent to dilatometer soundings and analyzing the test results. Five sites close to the University of Florida campus were chosen and dilatometer, piezoblade, cone penetration tests and sampling performed. This chapter describes the sites, locates and numbers all the tests performed at each site and briefly describes some of the testing procedures.

4.2 Test Sites

4.2.1 Location

Five test sites were chosen in Alachua County, Florida, within short driving distances of the University of Florida campus. Four of the sites bounded Lake Alice which is located on the edge of the University campus. The fifth site was at Lake Wauberg which is owned by the University for recreational purposes and is located about 12 miles south of the campus. The sites were chosen near the two lakes as it was desirable to have high groundwater levels. For the piezoblade tests, in order to maintain a fully saturated measuring device, it is necessary to initially drill to below the water table before inserting the probe into the hole. It was desirable that this be done by hand augering. At the sites chosen, the depth to the water table varied between 50 and 200 cm, and with distances from the water's edge.

Figure 4-1 shows the location of Alachua County within the state of Florida, and also the city of Gainesville, in which the University of Florida is situated. Figure 4-2 is a detail of a map and shows the locations of Lake Alice and Lake Wauberg.

4.2.2 Site Geology

Alachua County lies in the north-central portion of peninsular Florida in the coastal plane physiographic province (Wiener, 1982). The entire county is underlain by limestone of the Ocala Group of late Eocene age. A thick sequence of Hawthorn Formation clays and sands overlie the limestone in eastern Alachua County but are intermittent or nonexistent in the central and western portions of the county. A thin cover of loose surficial sands overlies the Hawthorn Formation. In western Alachua County a thick sequence of pleistocene dune sands overlies the Ocala Formation. The Hawthorn Formation consists of varying amounts of clay, quartz sand, and limestone with

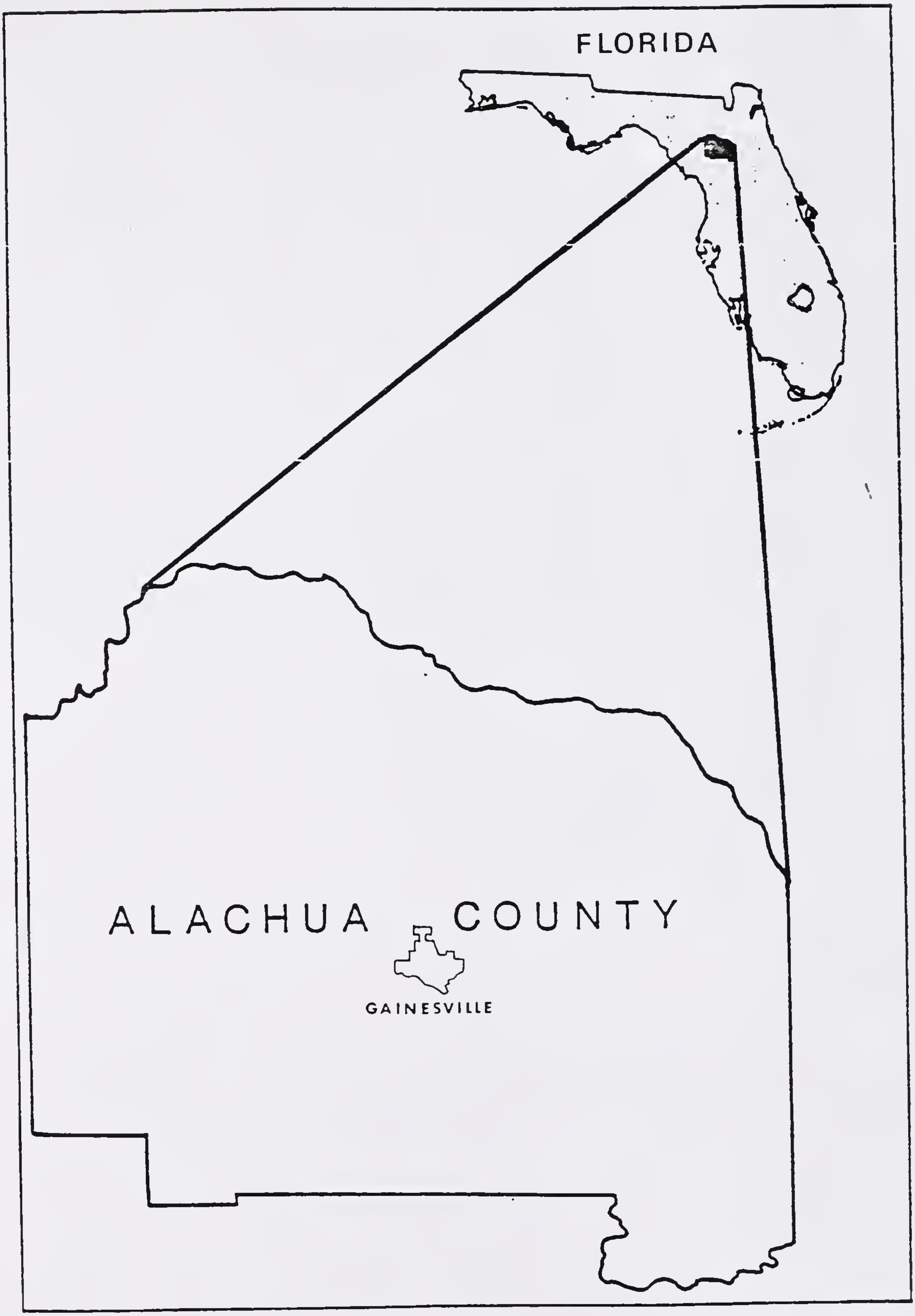


Figure 4-1. Location of Alachua County and the City of Gainesville

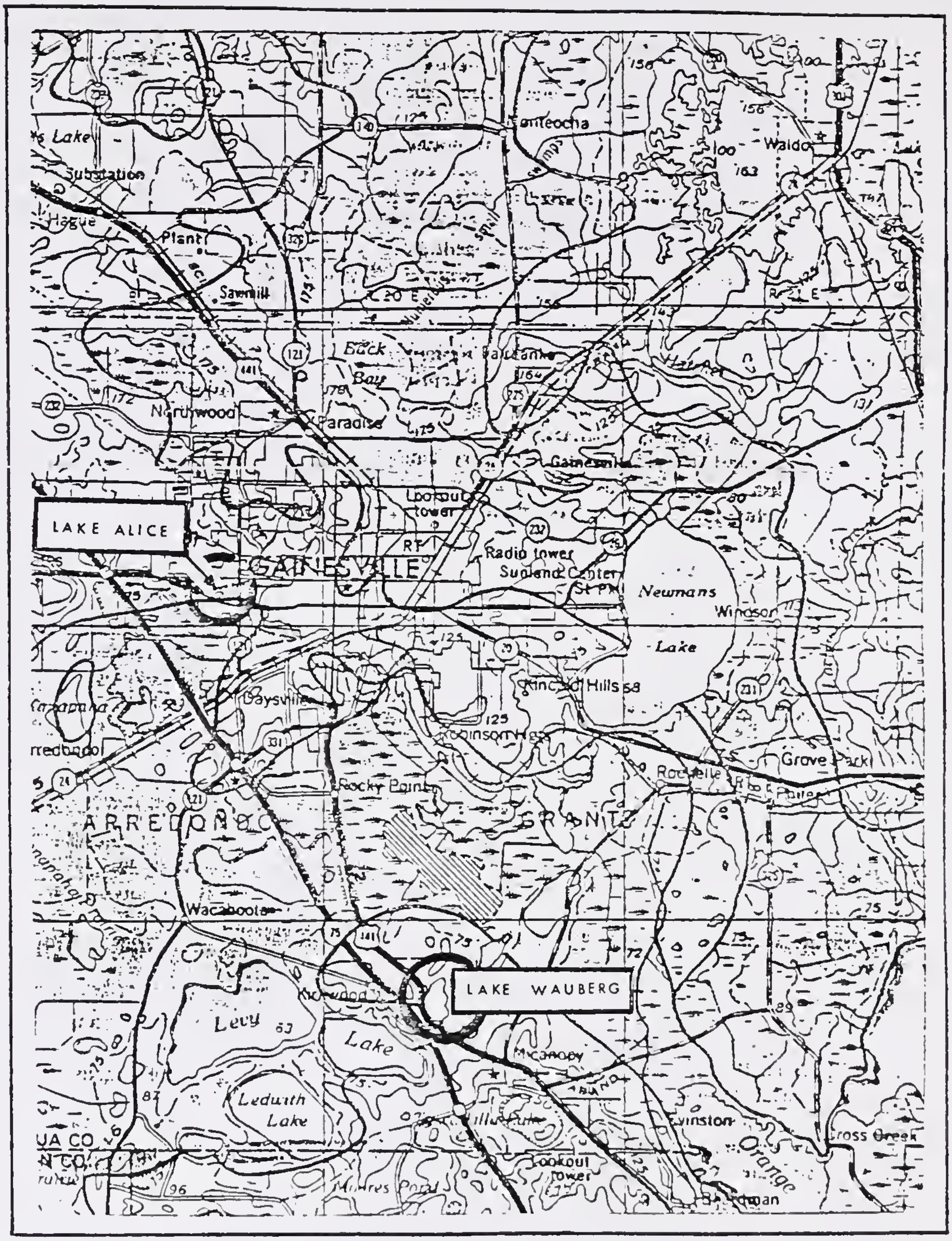


Figure 4-2. Location of Lake Alice and Lake Wauberg

phosphatic grains and pebbles. One of the most important characteristics of the Hawthorn Formation is its horizontal and vertical heterogeneity (Pirkle, 1956).

Lake Alice is located approximately in the center of Alachua County. This area consists of a nearly flat plain underlain by limestone of the Crystal River Formation and mantled by thin sandy soil and residual outlines of the Hawthorn Formation (Opper, 1981). Lake Alice is approximately 70 feet above sea level. Figure 4-3 shows a topographic map of the vicinity of the lake.

Lake Wauberg is located near the southern border of Alachua County and centrally in an East-West direction. The area around the lake consists of three different geological layers. The top layer is sand or a sand/clay mixture. According to Carl Opper (1981) the sand layer is from two to twelve feet thick with an average of about four feet, while the sand/clay layer is from five to seventeen feet in thickness. The clays consist of montmorillonite and kaolinite. Under this top layer is the Hawthorn clay which is a very dense plastic and very impermeable layer. The Hawthorn also contains many lenses of sand and limestone.

Lying below the Hawthorn Formation on an unconformable surface are several thousand feet of limestone and dolomite, the Ocala limestone. Figure 4-4 shows a topographic map of the Lake Wauberg area. Figure 4-5 shows a block perspective diagram of an Ocala Limestone structure contour

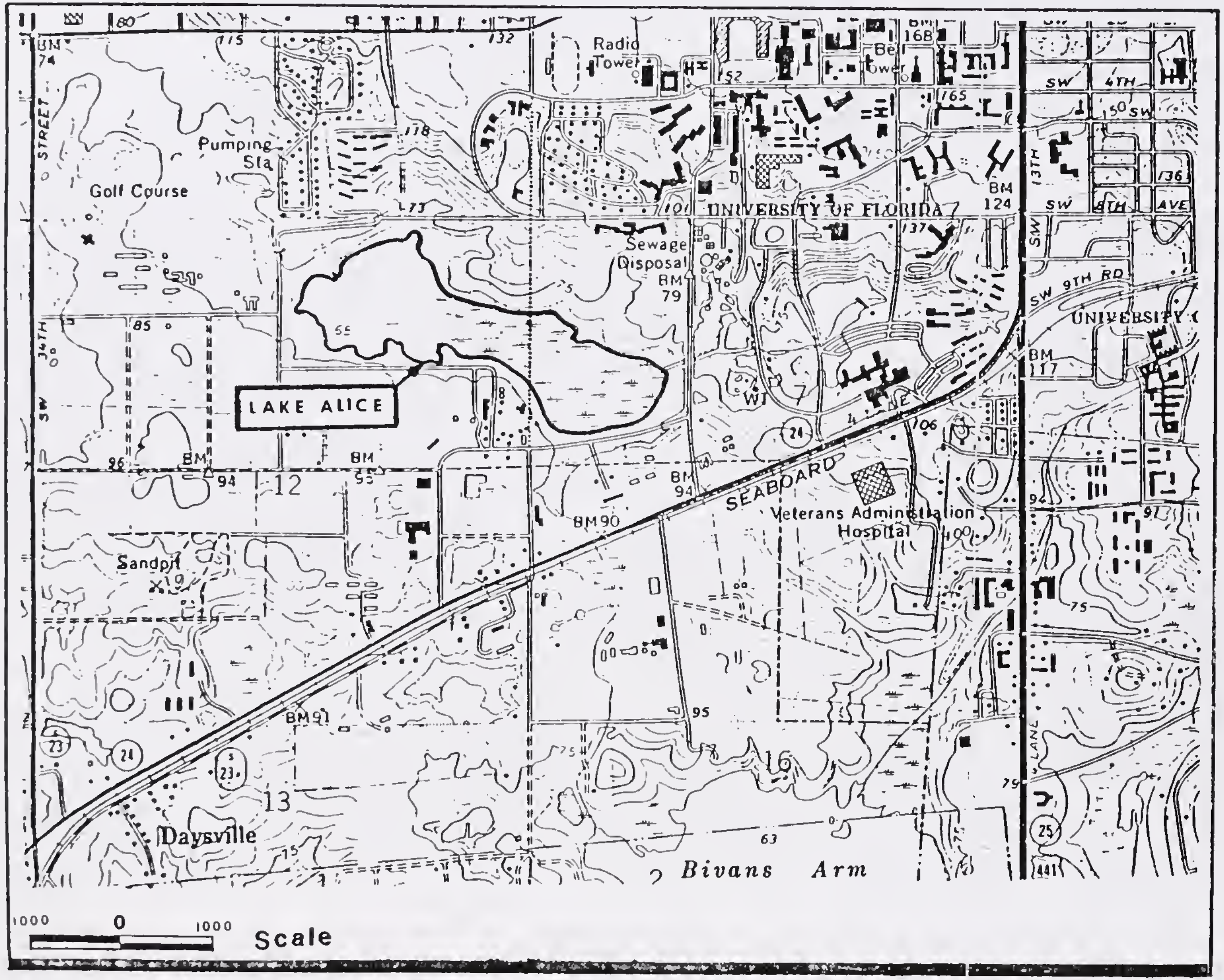


Figure 4-3. Topographic Map of Lake Alice

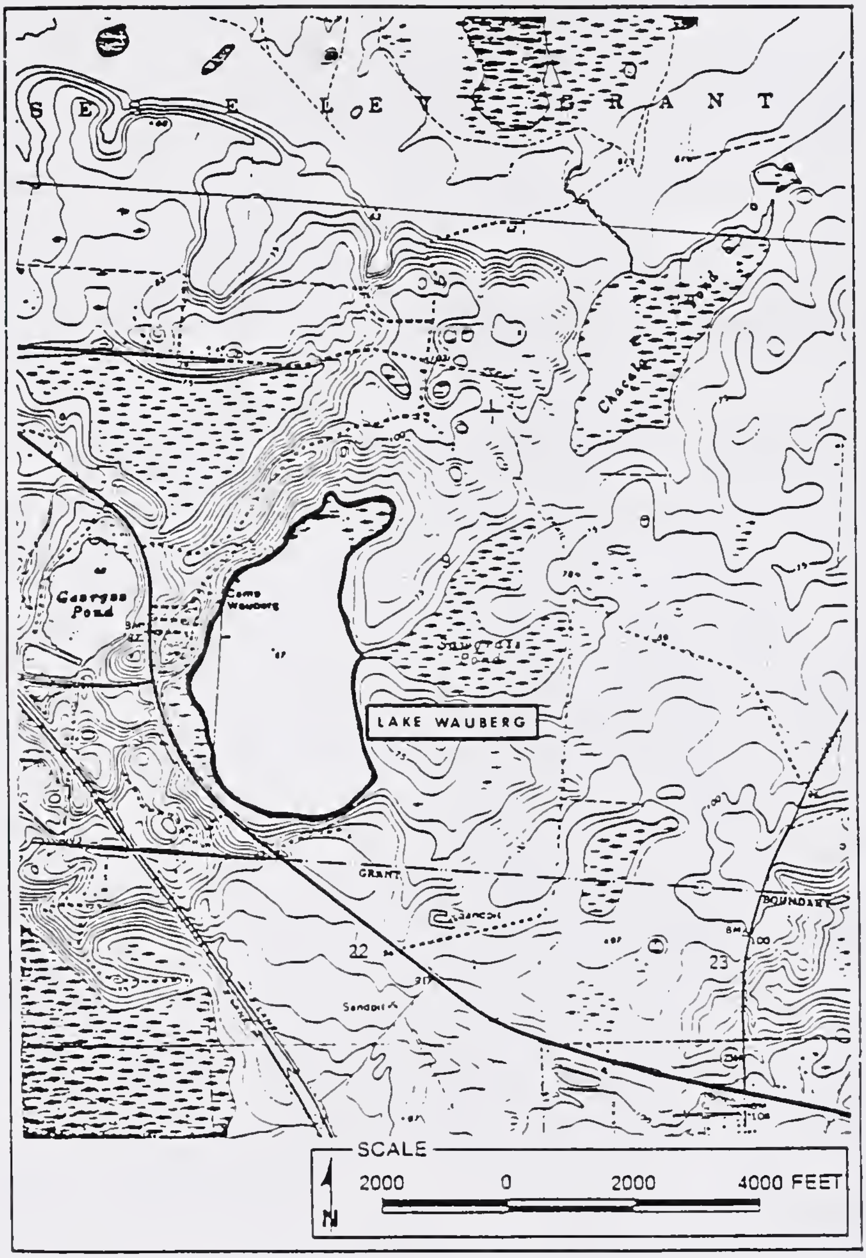


Figure 4-4. Topographic Map of Lake Wauberg

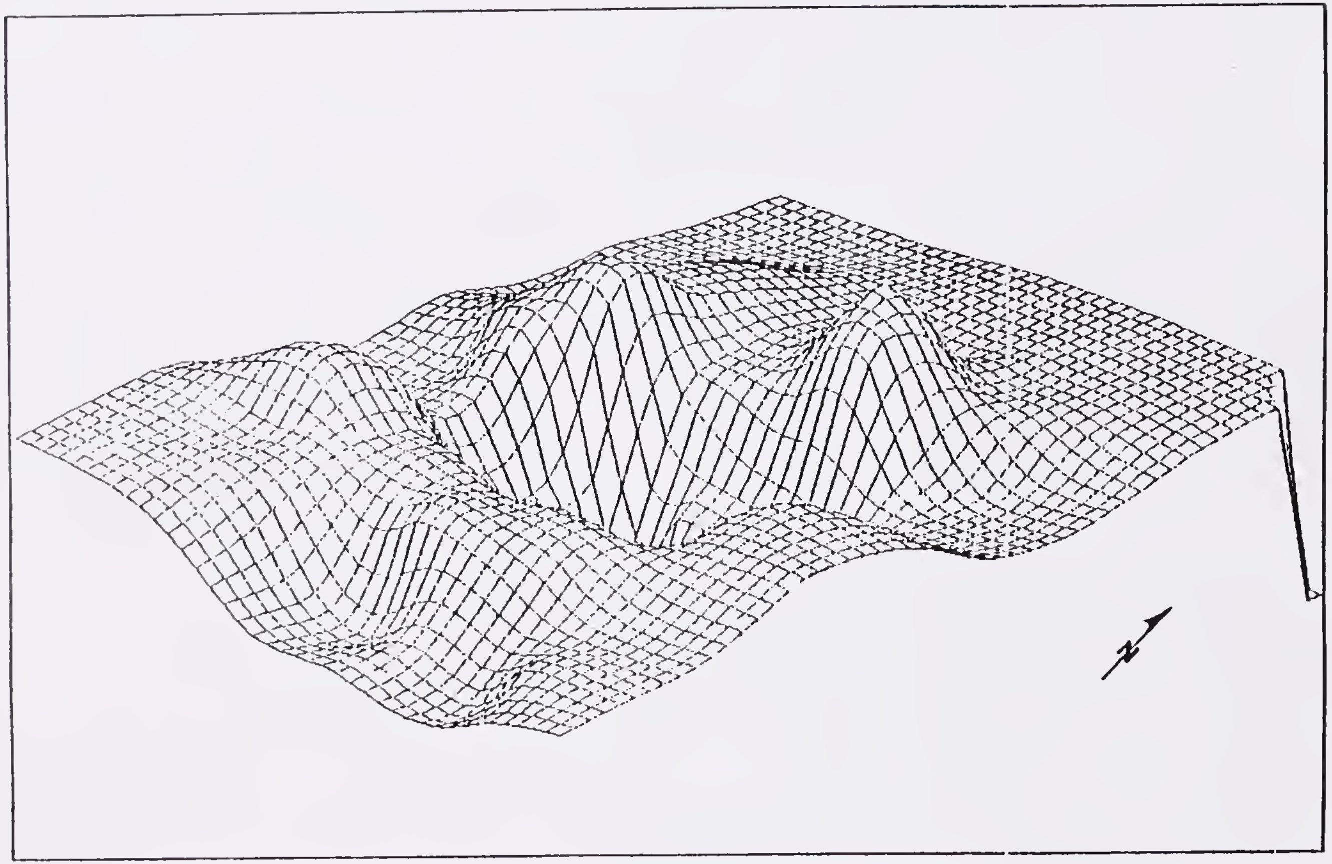


Figure 4-5. Block Perspective Diagram of Ocala Limestone Structure Contour Map. (Wiener, 1982)

map produced by a computer. This illustrates that the elevation of the top of the rock is very variable and changes rapidly.

4.2.3 Site Plans

Figures 4-6 to 4-10 contain plans of the five test sites. For each site, symbols indicate the location and type of test performed. Distances are in general marked from the water's edge. The boring numbers are indicated beside the symbols. These allow the profiles and test results of future sections to be related back to their field locations. For example, the following tests were performed at Site 3, Figure 4-8; three piezoblade soundings, numbers 4, 5, and 13; three dilatometer soundings, numbers 4, 5, and 12; four cone penetration tests, numbers 3, 9, 10, and 11; two disturbed sampling borings, numbers 3 and 4; and one undisturbed sampling boring, number 2. The piezoblade profiles can be found in Appendix A, the dilatometer results in Appendix B, the cone penetration test results in Appendix C, consolidation test plot from undisturbed samples in Appendix D, triaxial test plots in Appendix E and soil classifications from disturbed samples in Chapter 5.

4.3 Piezoblade Testing

4.3.1 Test Preparation

The piezoblade, de-aired and saturated as described in Section 2.7, was brought to the site enclosed in a water

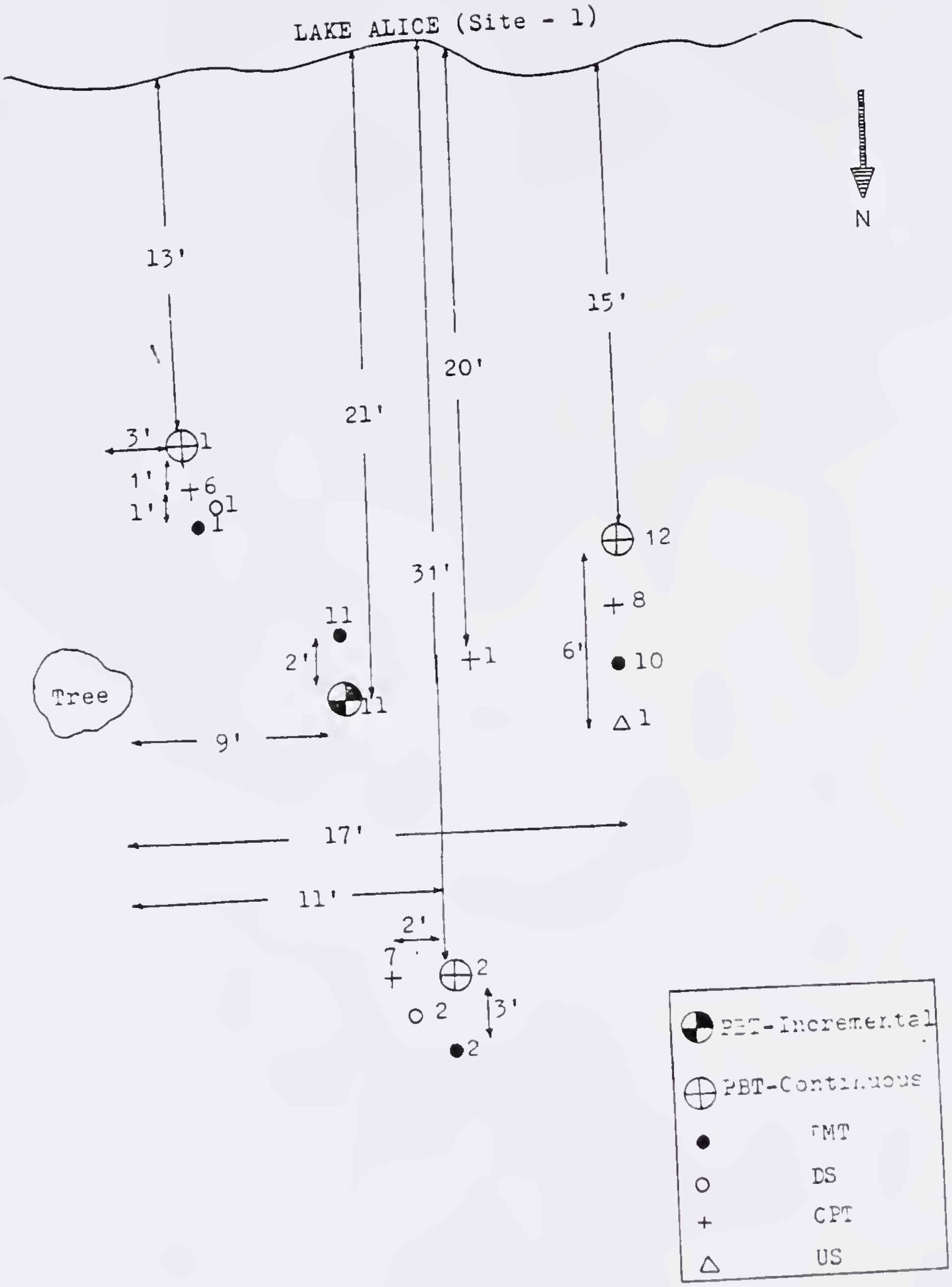


Figure 4-6. Site-1, North of Lake Alice

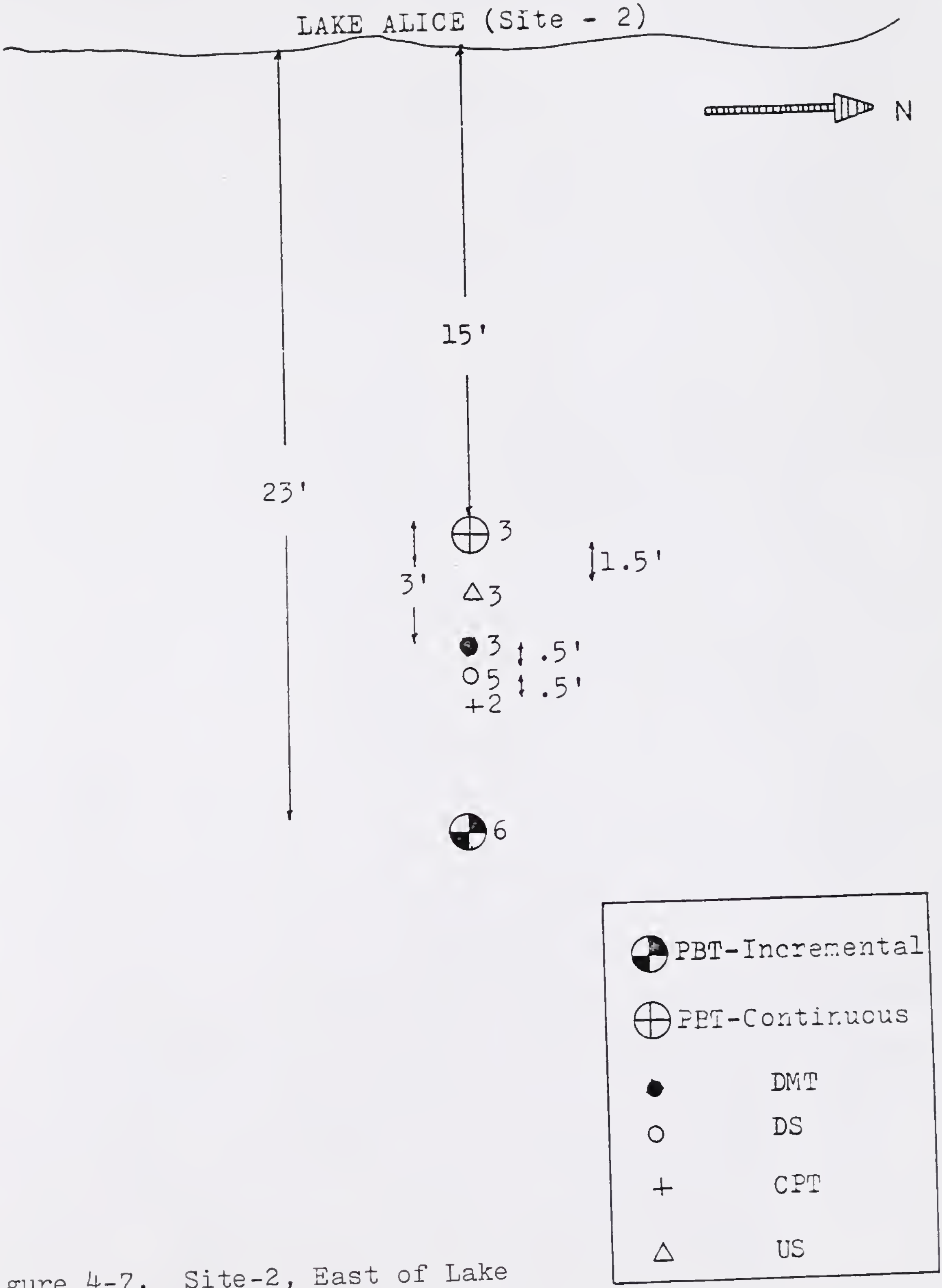


Figure 4-7. Site-2, East of Lake Alice

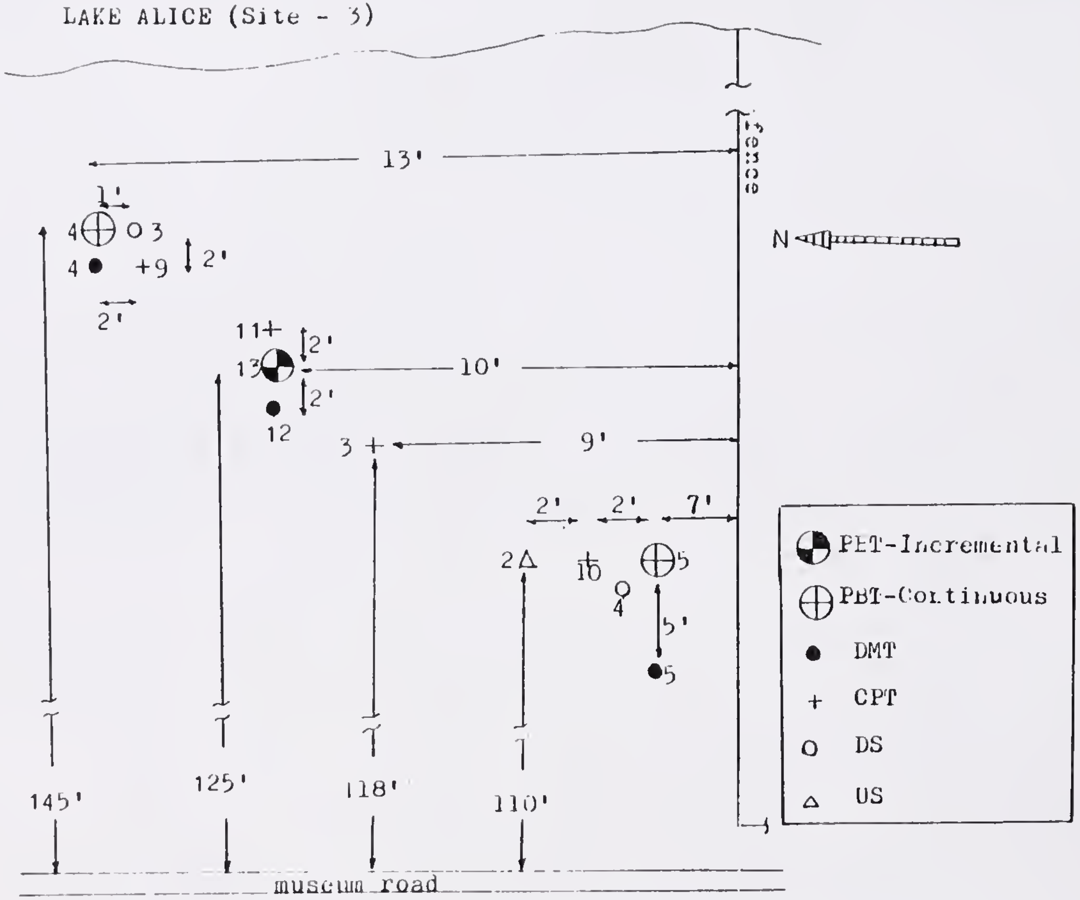


Figure 4-8. Site-3, West of Lake Alice

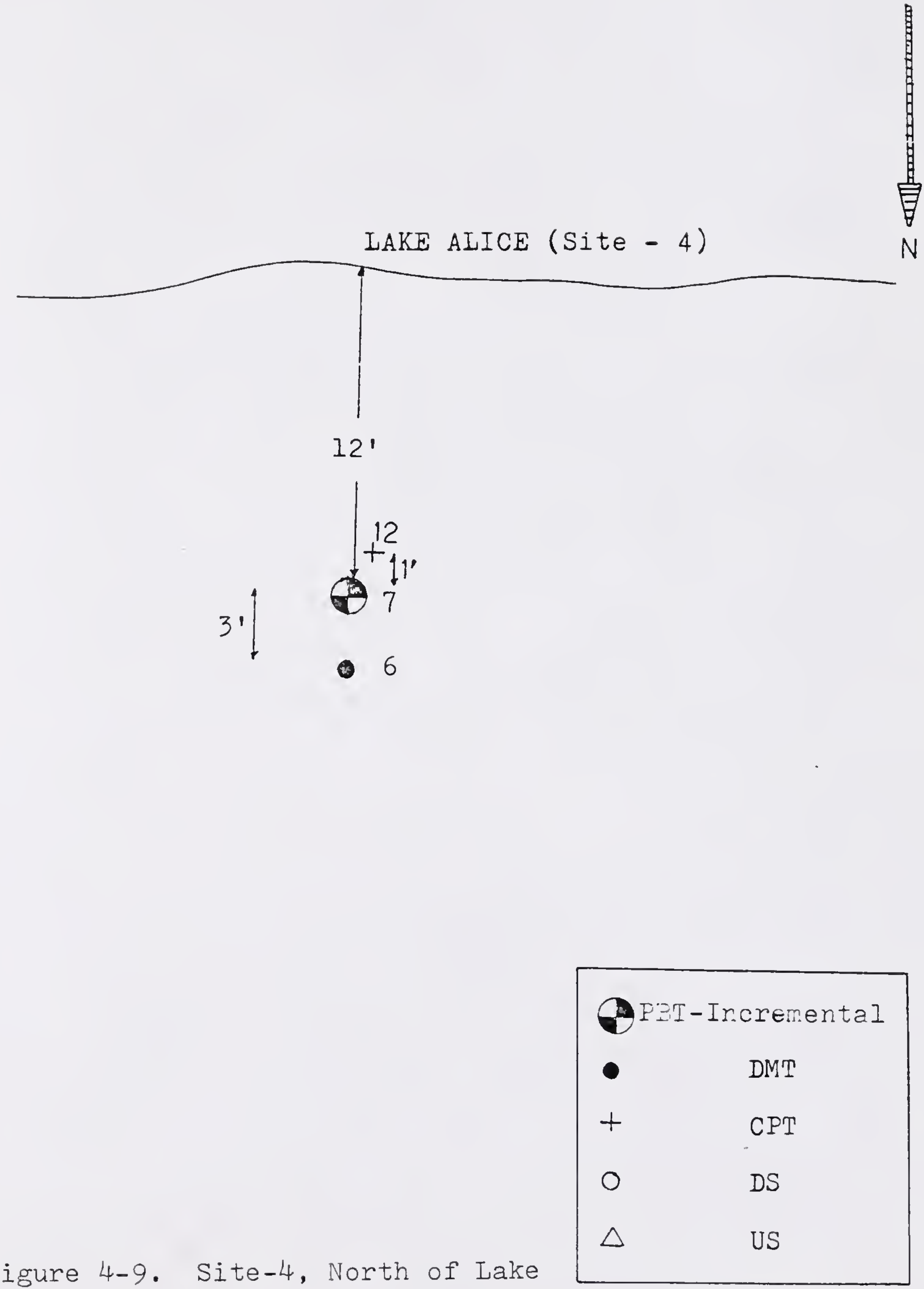


Figure 4-9. Site-4, North of Lake Alice

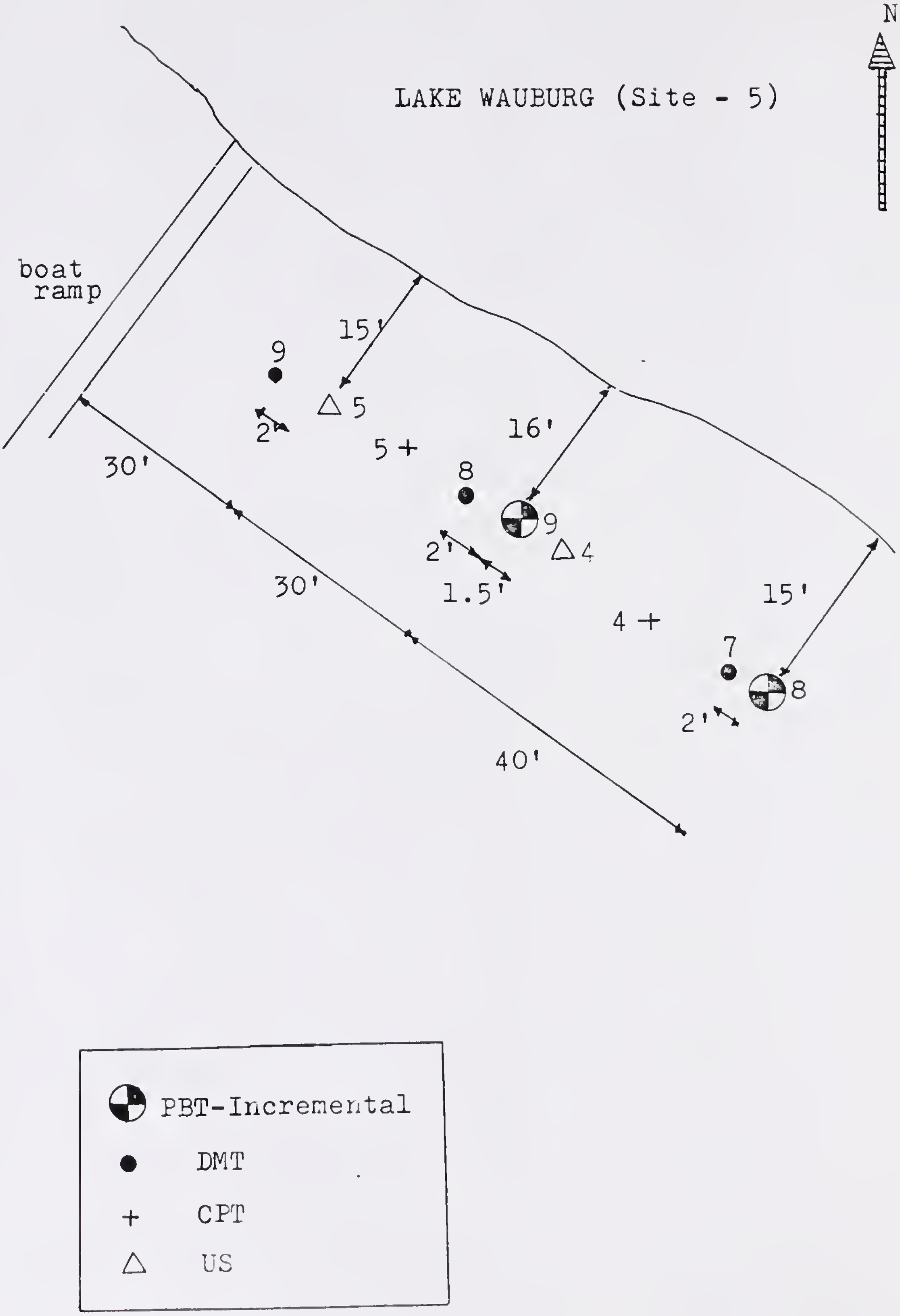


Figure 4-10. Site-5, Northwest of Lake Wauberg

filled plastic bag and submerged in a bucket of water. The test hole was hand augered to about 30 cm below the water table and the truck located over the hole. The bucket and piezoblade were then placed adjacent to the hole under the truck, and the 100 foot cable passed up through the hole in the floor of the truck. The cable was threaded through the appropriate number of 1 m drill rods, and the four conducting wires connected to the Clamp Box. The remaining control and recording instruments were hooked up as described in Section 2.5, and the excitation voltage set to 15 volts.

The piezoblade was then removed from the bucket, screwed to the first length of drill rod and lowered slowly into the test hole. Additional rods were added until the probe was sitting underwater at the bottom of the hole. It was then pushed about 10 cm into the soil, withdrawn the 10 cm and this process repeated a number of times to ensure that the plastic bag was torn and the blade was in contact with the natural groundwater. The depth of the piezoblade below the ground surface was accurately measured and the test was then ready to proceed.

4.3.2 Types of Piezoblade Tests

Two different types of piezoblade tests were performed, continuous penetration and incremental penetration. In the continuous test the blade was pushed into the ground at a constant rate of 2 cm/sec for the entire 1 m penetration length of the rods. Only when an additional length of rod needed to be added was the test penetration stopped. A continuous plot of generated excess pore water pressure versus depth was thus obtained for each sounding. Since the recorder speed was constant, linear interpolation on the chart over each meter length of depth was possible. Millivolt readings at 5 cm ground depth increments were taken and using the correlations of Figure 2-25, pore pressures determined and plotted. In Table 4-1 the continuous penetration tests are indicated along with the page numbers where test plots can be found.

In the incremental test, a penetration rate of 2 cm/sec was again used but insertion was stopped every 20 cm and the excess pore water pressures allowed to dissipate for up to two minutes. This was done to simulate the dilatometer test, in which tests are performed at 20 cm increments and take about one minute each to perform. Table 4-1 also lists these tests and where the plots may be found.

4.4 Dilatometer Testing

After calibrating the dilatometer as described in Section 3.2.2, the cable was stretched out in a straight line and the desired number of drill rods run onto it. The cable quick connect was then inserted into the control unit. The pressure source was also connected to the control unit and set to a pressure less than the maximum capacity of

Table 4-1. List of Piezoblade Tests Performed

Test No.	Site No.	Test Type *	Depth (m)	Date Tested	Test Results On Page
1	1	CP	5.00	07/01/81	192
2	1	CP	9.00	07/06/81	193
3	2	CP	8.00	07/09/81	194
4	3	CP	7.10	07/17/81	195
5	3	CP	9.00	07/21/81	196
6	2	IP	3.20	08/01/81	203
7	4	IP	2.00	08/03/81	204
8	5	IP	4.80	08/14/81	205
9	5	IP	6.20	08/15/81	206
11	1	IP	5.20	08/17/81	207
12	1	CP	8.00	09/22/81	201
13	3	IP	3.50	09/24/81	208

*CP=Continuous Penetration
IP=Incremental Penetration

the unit. Originally carbon dioxide was used but some freezing of the regulators was found, so a change was made to nitrogen gas. The zero pressure Z_M, the reading on the control unit gauge at zero pressure, was recorded.

The first lengths of drill rod were attached and the dilatometer hung from the hydraulic ram just above the ground surface. The ground wire, which completes the electrical circuit, was connected to the control box and by an alligator clip to the drill rods. The blade was then penetrated into the ground at a constant rate of 2 cm/sec in 20 cm increments and testing performed as described in Section 3.2.2. The micrometer valve which controls the gas passing to the dilatometer was opened slowly, ideally taking 15 seconds to read pressure A, membrane lift-off, and a further 15 seconds to reach pressure B, the 1 mm membrane expansion. On reaching pressure B, the vent valve was immediately opened and finally the micrometer valve closed. The dilatometer was then ready for penetration to the next test depth. A computer program was used to reduce and plot the field data. Table 4-2 lists the dilatometer tests performed and the pages on which the results can be found.

4.5 Cone Penetration Testing

Cone penetration tests, with which the University of Florida Geotechnical group has considerable experience, were performed at all sites. The test equipment has

Table 4-2. List of Dilatometer Tests Performed

Test No.	Site No.	Depth (m)	Date Tested	Test Results On Page
1	1	5.00	07/01/81	210-221
2	1	8.60	07/06/81	211-222
3	2	7.00	07/09/81	212-223
4	3	7.40	07/20/81	213-224
5	3	8.80	07/20/81	214-225
6	4	1.60	08/03/81	215-226
7	5	4.80	08/14/81	216-227
8	5	5.80	08/15/81	217-228
9	5	4.60	08/15/81	218-229
10	1	7.80	10/23/81	219-230
11	1	7.80	10/24/81	220-231

already been briefly described in Section 3.3.1. In the mechanical system employed during this research, inner and outer rods are used to transmit the loads to the tip. Inner rods which had been removed for piezoblade and dilatometer testing were lubricated and replaced within the outer rods. A friction reducer was used on the first rod above the tip. Tests were all performed in the standard manner, with a 2 cm/sec penetration rate and readings taken every 20 cm. Table 4-3 lists the cone penetration tests performed and the location in this dissertation of the test results.

Table 4-3. List of Cone Penetration Test Soundings Performed

Test No.	Site No.	Depth (m)	Date Tested	Test Results On Page
1	1	9.80	07/07/81	233
2	2	7.40	07/07/81	234
3	3	8.40	07/21/81	235
4	5	5.80	08/15/81	236
5	5	6.00	08/15/81	237
6	1	8.80	09/23/81	238
7	1	8.80	09/23/81	239
8	1	8.80	09/23/81	240
9	3	7.00	09/24/81	241
10	3	7.80	09/25/81	242
11	3	8.60	09/24/81	243
12	4	1.60	06/16/82	244

CHAPTER 5

LABORATORY SOIL TESTS

5.1 Introduction

Samples were obtained from the five field test sites for laboratory testing. Twenty-five disturbed samples were collected from five hand augered borings to a maximum depth of 3 m. Nine undisturbed piston samples, the deepest to 6.1 m, were taken from five other borings.

Only classification tests were performed on the disturbed samples. These included determination of moisture content, specific gravity, Atterberg liquid and plastic limits and grain size distributions. Consolidation and triaxial tests were performed on the undisturbed samples as well as the above classification tests.

The primary purpose of this testing program was to provide a comparison between laboratory determined and dilatometer determined soil parameters.

5.2 Sampling Procedures

The disturbed samples were obtained using the 5.72 cm diameter, 20 cm long bucket hand auger shown in Figure 5.1. This sampler, which was designed for use with the Menard pressuremeter, can be used to depths of about 5 m under ideal conditions. The rods are conically threaded and come in one meter lengths. In this testing program samples at 50 cm intervals were collected, placed in glass jars and sealed.

The undisturbed samples were obtained using the Swedish Fixed Piston Sampler shown in Figure 5.2. This sampler is pushed into the ground using the cone penetration test truck. At the desired sampling depth, a steel tape is passed down the string of drill rods to engage a bayonet connection. This tape, which is fixed to the truck, holds the piston while the sampler is further penetrated. At the end of the stroke the tape is automatically disengaged. The sampler is then withdrawn and the sample in four, 17 cm long, 50 mm I.D., 53 mm O.D. fiberglass liners extruded. The liners are capped for transportation to the laboratory. Table 5-1 locates the nine piston samples taken, by boring number, site, and depth.

5.3 Tests on Disturbed Samples

The tests performed on the disturbed samples were all of the classification variety. Water contents were determined for all samples immediately on return to the laboratory. Specific gravity tests were also performed on all samples while Atterberg liquid and plastic limits were determined for those samples exhibiting cohesion. Before sieving, each sample was washed through a #200 sieve to separate sandy material from the fines. Both

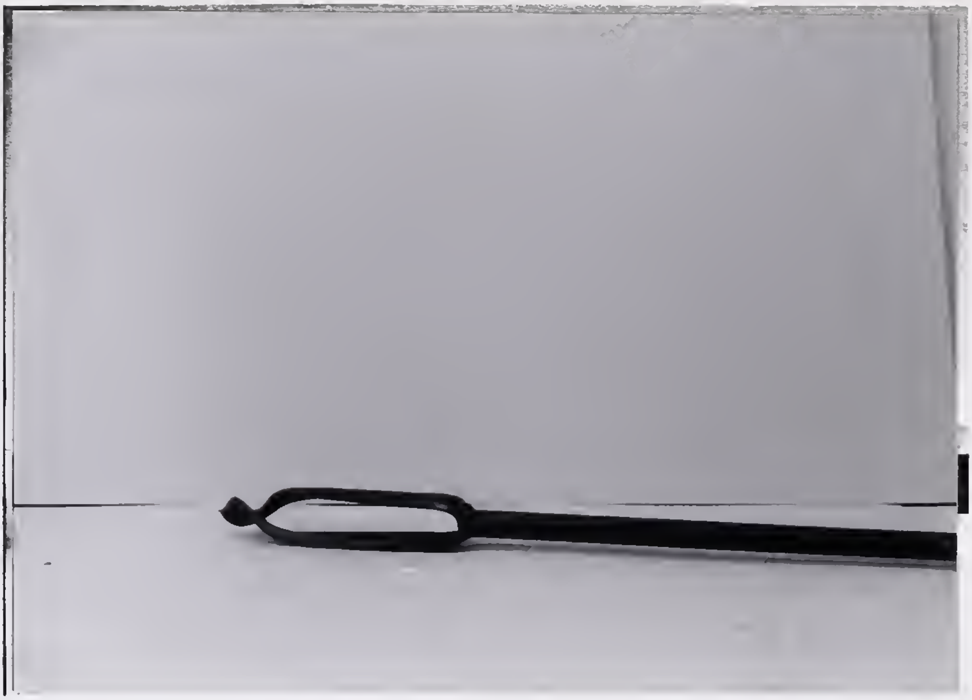


Figure 5-1. The Hand Auger Sampler

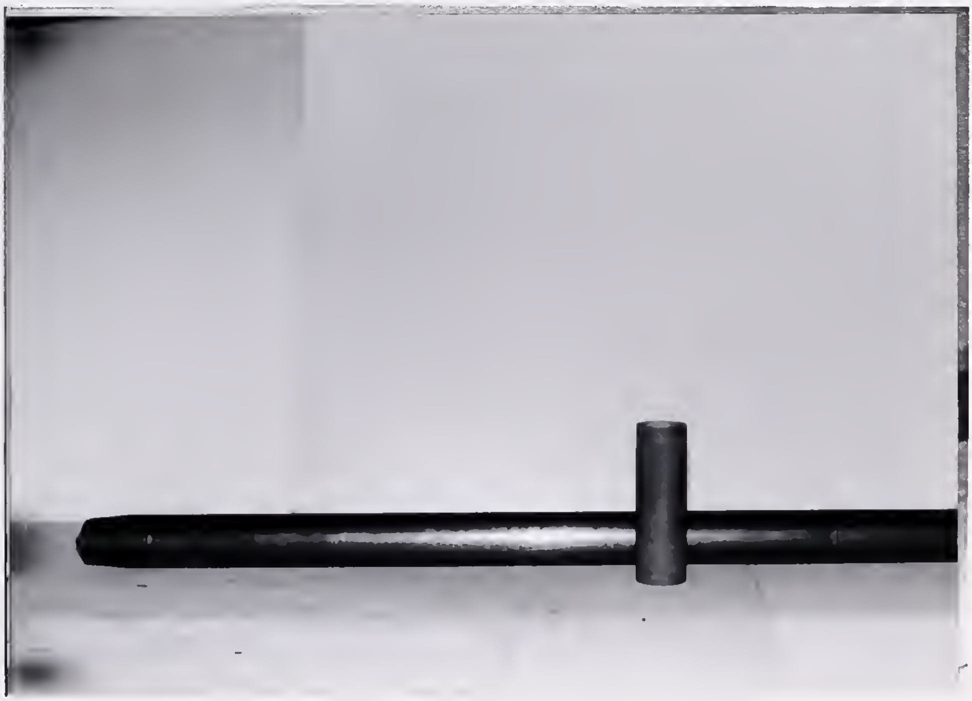


Figure 5-2. The Swedish Fixed Piston Sampler

Table 5-1. List of Undisturbed Samples Taken

Number	Site No.	Sample No.	Sample Depth (m)	Date Obtained
1	1	1	1.50 - 2.17	08/10/81
		2	3.45 - 4.12	08/10/81
		3	4.50 - 5.18	08/12/81
2	3	4	4.40 - 4.80	09/24/81
		5	5.50 - 6.00	09/25/81
3	2	6	3.45 - 4.20	09/28/81
		7	5.40 - 6.10	06/16/82
4	5	8	3.40 - 4.10	07/15/82
5	5	9	3.40 - 4.10	07/15/82

fractions were then oven-dried and a sieve analysis later performed on the coarse fraction. For any sample having more than 10% by weight passing the #200 sieve, a hydrometer analysis was also performed. Each soil was then classified according to the Unified Soil Classification System.
All laboratory tests were carried out according to the acceptable standard procedures described in Lambe (1951) and Bowles (1970).

Results from the disturbed sample tests and their classification are summarized in Tables 5-2 through 5-6.

5.4 Tests on Undisturbed Samples

One dimensional consolidation and consolidated undrained triaxial tests were performed on the undisturbed samples as well as the routine classification tests described above. A total of fifteen consolidation tests and nine triaxial tests were performed.

5.4.1 Consolidation Tests

The consolidation tests were carried out in two different pieces of equipment, the Anteus and the Soil Test consolidometers. Standard test procedures were followed in all tests and are not described herein. The Anteus equipment, shown in Figure 5-3, permits back saturation of the sample. Drainage was from the top surface only and load increments of 0.5, 1.0, 2.0, 4.0, 8.0, and 16.0 tons/ft.^{2} were used. The ring had a diameter of 50.3 mm and the sample height was 19.0 mm.

Table 5-2. Results of Tests on Disturbed Sample from Boring DS-1

Depth (cm)	Water Content w%	Specific Gravity G_s	Liquid Limit LL	Plastic Limit PL	Plastic Index PI	% -200 sieve	C_u	Soil Type	Classification
50	11.11	2.66	--	--	0	4.81	2.56	sand	SP
100	20.35	2.66	--	--	0	11.96	6.4	sand	SW-SM
150	33.84	2.64	40.15	27.10	13.05	33.30	>6	silty sand	SM
200	31.08	2.65	47.50	26.08	21.42	30.94	>6	silty sand	SM

LOCATION. North of Lake Alice BORING NO. DS-1

Table 5-3. Results of Test on Disturbed Samples from Boring DS-2

Depth (cm)	Water Content w%	Specific Gravity G_s	Liquid Limit LL	Plastic Limit PL	Plastic Index PI	% -200 sieve	C_u	Soil Type	Classification
50	5.70	2.65	--	--	0	9.31	2.88	sand	SP-SM
100	20.28	2.65	23.85	16.97	6.88	23.36	>6	silty clayey sand	SM-SC
150	18.51	2.70	--	--	0	14.38	11.2	silty sand	SM
200	18.86	2.67	--	--	0	17.83	14.0	silty sand	SM
250	25.61	2.70	--	--	0	18.46	7.81	silty sand	SM
300	28.21	2.73	32.58	21.67	19.91	20.45	12.73	clayey sand	SC

LOCATION. North of Lake Alice BORING NO.DS-2

Table 5-4. Results of Test on Disturbed Sample from Boring DS-3

Depth (cm)	Water Content w%	Specific Gravity G_s	Liquid Limit LL	Plastic Limit PL	Plastic Index PI	% -200 sieve	C_u	Soil Type	Classification
50	6.02	2.65	--	--	0	2.62	2.50	sand	SP
100	10.04	2.67	--	--	0	2.09	2.29	sand	SP
150	19.44	2.67	--	--	0	3.55	3.20	sand	SP
200	20.89	2.65	--	--	0	3.20	2.60	sand	SP
250	21.03	2.68	--	--	0	3.02	2.80	sand	SP

LOCATION. West of Lake Alice BORING NO. DS-3

Table 5-5. Results of Tests on Disturbed Samples from Boring DS-4

Depth (cm)	Water Content w%	Specific Gravity G_s	Liquid Limit LL	Plastic Limit PL	Plastic Index PI	% -200 sieve	C_u	Soil Type	Classification
50	5.57	2.65	--	--	0	3.63	2.80	sand	SP
100	5.16	2.65	--	--	0	2.71	3.00	sand	SP
150	9.16	2.70	--	--	0	3.06	2.80	sand	SP
200	18.70	2.68	--	--	0	4.26	3.05	sand	SP
250	20.27	2.66	--	--	0	4.34	3.47	sand	SP
300	21.15	2.66	--	--	0	4.57	3.80	sand	SP

LOCATION. West of Lake Alice BORING NO. DS-4

Table 5-6. Results of Tests on Disturbed Samples from Boring DS-5

Depth (cm)	Water Content $w_{\%}$	Specific Gravity G_s	Liquid Limit LL	Plastic Limit PL	Plastic Index PI	% -200 sieve	C_u	Soil Type	Classification
50	14.46	2.65	--	--	0	6.97	3.65	silty sand	SP-SM
100	20.77	2.67	23.35	18.84	4.51	20.50	>6	silty sand	SM
150	20.16	2.68	29.65	19.66	5.99	30.58	>6	silty sand	SM
200	19.41	2.70	33.00	17.91	15.09	33.51	>6	clayey sand	SC

LOCATION. East of Lake Alice BORING NO. DS-5

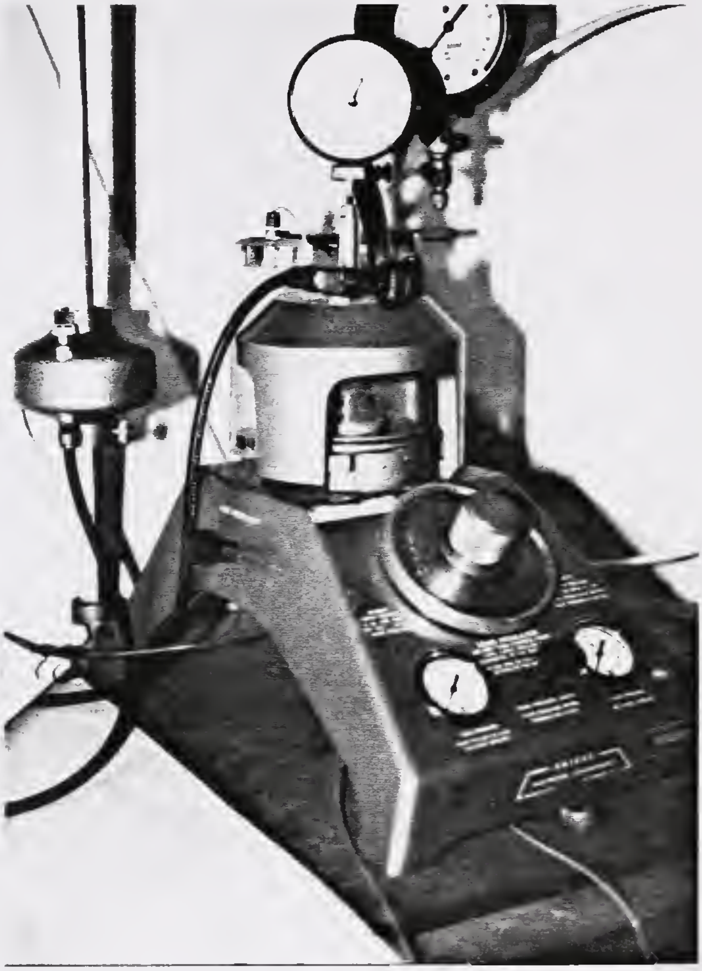


Figure 5-3. The Anteus Consolidometer

The Soil Test (Model C-221) consolidometer, Figure 5-4, does not permit back saturation but did allow both lower and higher (to 32tsf) loadings.

Results from the consolidation tests are summarized in Table 5-7 and the pages, where the laboratory test plots may be found, are indicated. Only in the Soil Test consolidometer tests was it possible to determine the constrained modulus M at the overburden pressure $\overline{P}_0$.

5.4.2 Triaxial Tests

Nine consolidated undrained triaxial tests were performed. Samples were extruded from their liners, trimmed top and bottom enclosed by membrane, accurately measured and weighed and placed in the triaxial chamber. Figure 5-5 shows the test set-up, consisting of triaxial cell, load cell, recorder and voltmeter.

The sample was then placed under a vacuum for 24 hours. To prevent disturbance of the sample, a vacuum of -12.7 psi was also applied to the chamber water. Thus, only an effective confining pressure of 2 psi acted on the sample. This was necessary since most samples were from a depth where the overburden pressure was less than 14.7 psi. After vacuuming, the sample was back pressured to dissolve all the remaining air in the voids. The back pressure was applied in very small increments until a pore pressure parameter B of one was achieved.

$$B = \Delta u / \Delta p$$

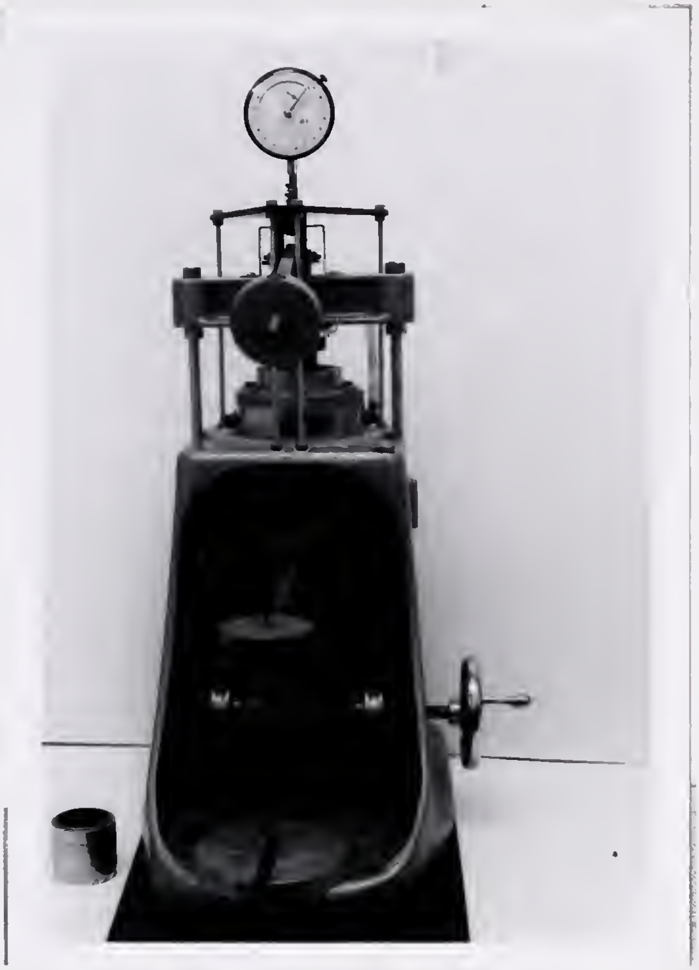


Figure 5-4. The Soil Test (Model C-221) Consolidometer

Table 5-7. Consolidation Test Results

Sample No.	Depth (cm)	$\overline{P}_0$ (TSF)	$\overline{P}_m$ (TSF)	OCR	M (TSF)	C_c	e - Log $\overline{P}$ on page	LogM - Log $\overline{P}$ on page
US-2 (352)	352	0.388	3.600	9.28	-	0.4754	246	-
US-3 (460)	460	0.489	5.600	11.50	-	0.2634	247	-
US-3 (495)	495	0.521	4.600	8.83	-	0.1797	248	-
UF-4 (440)	440	0.577	3.800	6.59	-	0.2850	249	-
US-5 (590)	590	0.698	1.975	7.92	-	0.1844	250	-
US-6 (380)	380	0.427	7.750	18.15	-	0.1761	251	-
US-7 (560)	560	0.600	4.550	7.58	185.00	0.4746	252	261
US-7 (580)	580	0.611	4.650	7.61	185.00	0.4495	253	262
US-7 (600)	600	0.631	4.700	7.45	239.00	0.4308	254	263
US-8 (360)	360	0.400	7.300	18.25	350.00	0.5973	255	264
US-8 (380)	380	0.416	7.200	17.31	252.57	0.7540	256	265
US-8 (400)	400	0.434	7.400	17.04	245.00	0.7987	257	266
US-9 (360)	360	0.400	19.000	47.50	680.00	1.1400	258	267
US-9 (380)	380	0.423	13.600	32.15	420.25	0.7232	259	268
US-9 (400)	400	0.445	20.090	45.18	390.00	0.7548	260	269

$\overline{P}0$ = effective overburden pressure $\overline{P}m$ = maximum effective past consolidation pressure

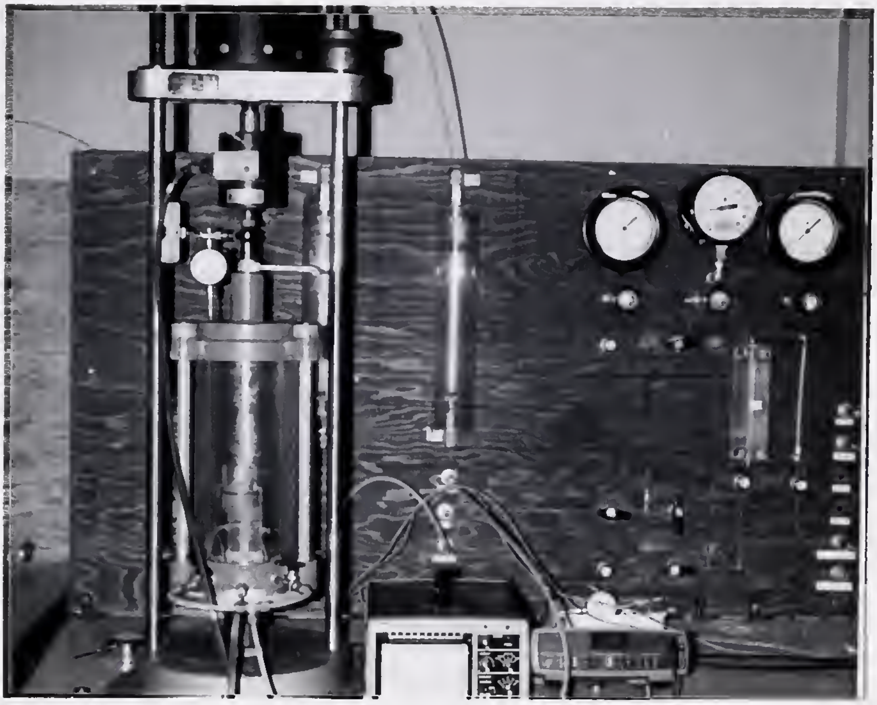


Figure 5-5. The Triaxial Test Apparatus

where

Δu = change in pore water pressure

Δp = change in confining pressure

The sample's effective overburden pressure was then applied and the sample allowed to consolidate for 24 hours.

The drainage line was then closed and a standard, constant rate of strain, undrained triaxial test performed with pore pressure measurement at the base. All tests showed an initial small positive pore pressure followed by a large negative pressure. Table 5-8 summarizes the test results and lists the pages on which the stress-strain and pore pressure-strain plots may be found. Results from the classification tests run on the undisturbed samples are included in Table 5-9.

Table 5-8. Triaxial Test Results

Sample No.	Depth (cm)	E (PSI)	Max. Deviator Stress (PSI)	ΔU (PSI) *	Stress-Strain on page	Pore Pressure-Strain on page
US-1 (200)	200	2500.00	31.36	-11.14	271	280
US-2 (363)	363	2000.00	9.55	-1.25	272	281
US-2 (378)	378	1923.00	15.96	-3.10	273	282
US-3 (467)	467	3000.00	25.57	-8.80	274	283
US-3 (502)	502	2631.58	30.61	-10.60	275	284
US-4 (465)	465	1851.85	28.72	-2.90	276	285
US-5 (575)	575	3566.67	41.66	-8.00	277	286
US-6 (388)	388	7128.57	179.47	-64.00	278	287
US-2 (394)	394	2083.33	12.00	--	279	-

* Excess pore pressure was measured at maximum deviator stress

Table 5-9. Laboratory Results from Undisturbed Samples

Sample No.	Site	Depth (m)	G_s	w%	% -200 sieve	LL	PL	PI	Classification
US-1 (200)	1	2.00	2.70	22.08	19.65	--	--	0	SM
US-2 (363)	1	3.63	2.82	56.59	85.47	140.10	46.52	93.58	CH
US-2 (378)	1	3.78	2.89	61.26	83.92	145.80	47.70	98.10	CH
US-2 (394)	1	3.94	2.81	58.07	89.63	137.40	45.92	91.48	CH
US-3 (455)	1	4.55	2.80	60.73	82.18	74.10	24.79	49.31	CH
US-3 (460)	1	4.60	2.74	29.91	68.05	41.98	24.30	17.68	CL
US-3 (467)	1	4.67	2.74	27.16	68.22	38.25	24.94	13.31	ML
US-3 (502)	1	5.02	2.73	24.97	71.48	40.50	17.02	23.48	CL
US-4 (440)	3	4.40	2.70	34.10	23.97	--	--	0	SM
US-4 (465)	3	4.65	2.72	29.90	66.29	87.50	51.37	36.13	MH
US-4 (480)	3	4.80	2.87	--	68.88	85.75	51.89	33.86	MH
US-5 (575)	3	5.75	2.70	33.18	41.12	43.50	29.27	14.23	SM
US-5 (590)	3	5.90	2.70	36.18	53.36	36.90	27.16	9.74	ML

Table 5-9. (Continued)

Sample No.	Site	Depth (m)	G_s	w%	% -200 sieve	LL	PL	PI	Classification
US-6 (355)	2	3.55	2.69	16.75	23.15	26.26	17.61	8.65	SC
US-6 (380)	2	3.80	2.65	17.22	26.23	25.02	17.47	7.55	SC
US-7 (560)	2	5.60	2.77	42.47	64.66	74.50	32.90	41.60	CH
US-7 (580)	2	5.80	2.78	37.83	59.75	71.25	33.08	38.17	CH
US-7 (600)	2	6.00	2.79	39.29	68.80	68.50	26.77	41.73	CH
US-8 (360)	5	3.60	2.72	67.15	94.82	133.13	47.06	86.07	CH
US-9 (360)	5	3.60	2.70	74.35	87.94	146.50	50.30	96.20	CH

CHAPTER 6

STEREO PHOTOGRAPHY STUDY

6.1 Introduction

A laboratory experiment was performed in which a dummy blade, identical in shape and size to the piezoblade and dilatometer, was penetrated into a container of dry sand. A technique of stereo photography was used to accurately measure the movement of the sand grains around the probe. From these displacements strains could be calculated. The purpose of the experimentation was two fold:

A. To determine the strains and thus, qualitatively, the disturbance around this particular shape of probe.

B. To obtain a comparison of this blade's disturbance with that previously determined around the cylindrical-cone shape of the standard cone penetration test tip.

Tests were performed in five different density sands using techniques and equipment identical to those used in the earlier cone tip studies.

In conventional aerial stereo photography methods, two photographs of an object are taken from slightly different camera positions. The photographs are then placed in a stereo projector and the stereo image viewed.

This is illustrated in Figure 6-1. The height of the object is due to the difference in locations of corresponding points in the two photographs. A similar stereo effect can be achieved by fixing the camera and taking two photographs of a displaced object. In this case the height of the image is a measure of the displacement of the object in a direction parallel to the line joining the projectors, Figures 6-2 and 6-3. The normal component of movement can be measured by rotating the photographs 90°, and the actual displacements calculated as the vector sum of these components. This technique, called pseudo stereo photogrammetry, was pioneered at the University of Southampton, Butterfield et al.(1970), Andrawes and Butterfield (1973).

In the current study, a dilatometer shaped probe was pushed into a container of sand, directly adjacent to the one glass wall of the box, Figure 6-4. Photographs were taken from a fixed camera, one prior to penetration and several at different penetration depths. By viewing any two photographs in stereo, as illustrated above, grain movement could be determined at any point in the sand against the glass wall.

6.2 Equipment

The equipment used in this research is essentially the same as used in the earlier cone studies, Davidson (1980a), Davidson et al. (1981).

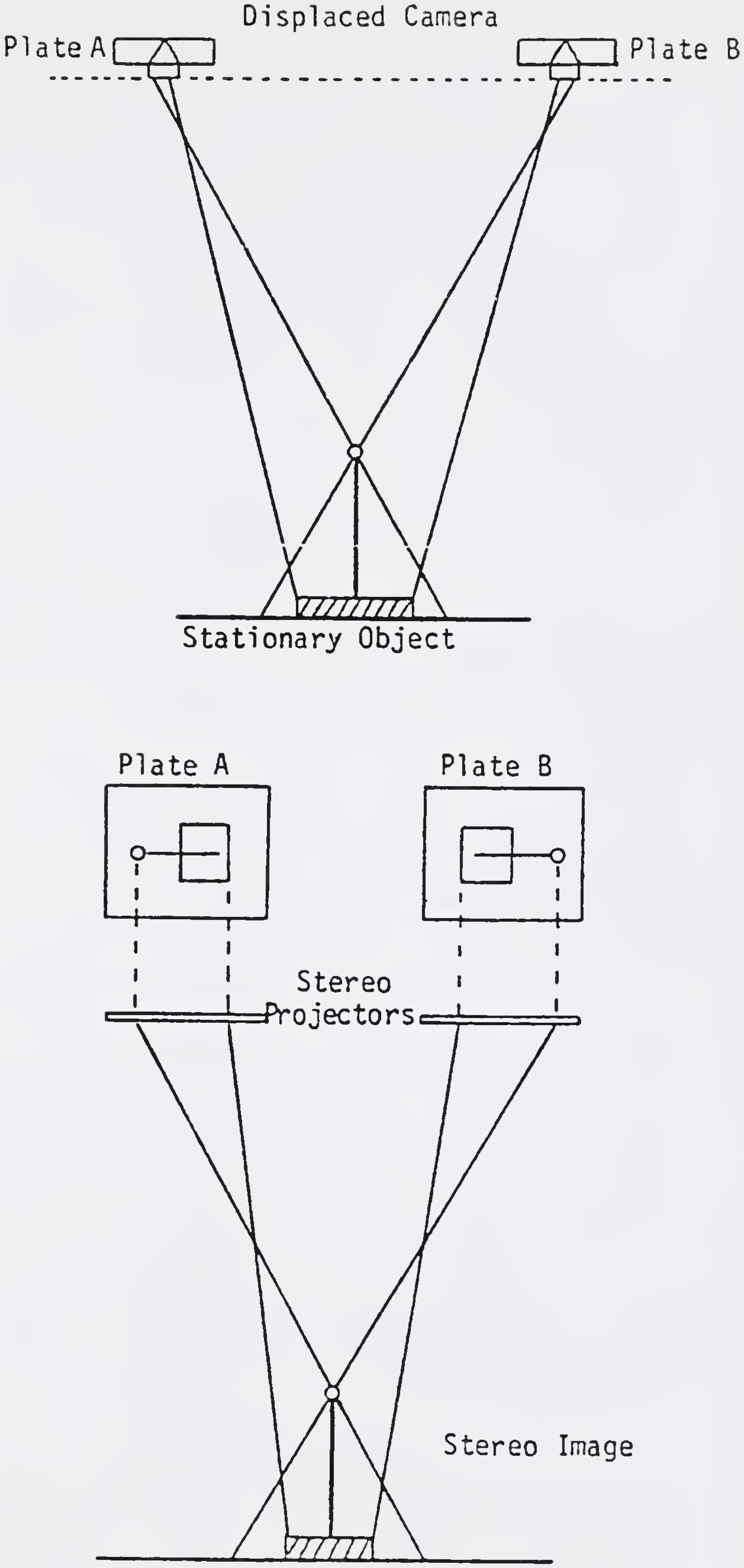


Figure 6-1. Stereo Image-Displaced Camera (After Andrawes, 1976)

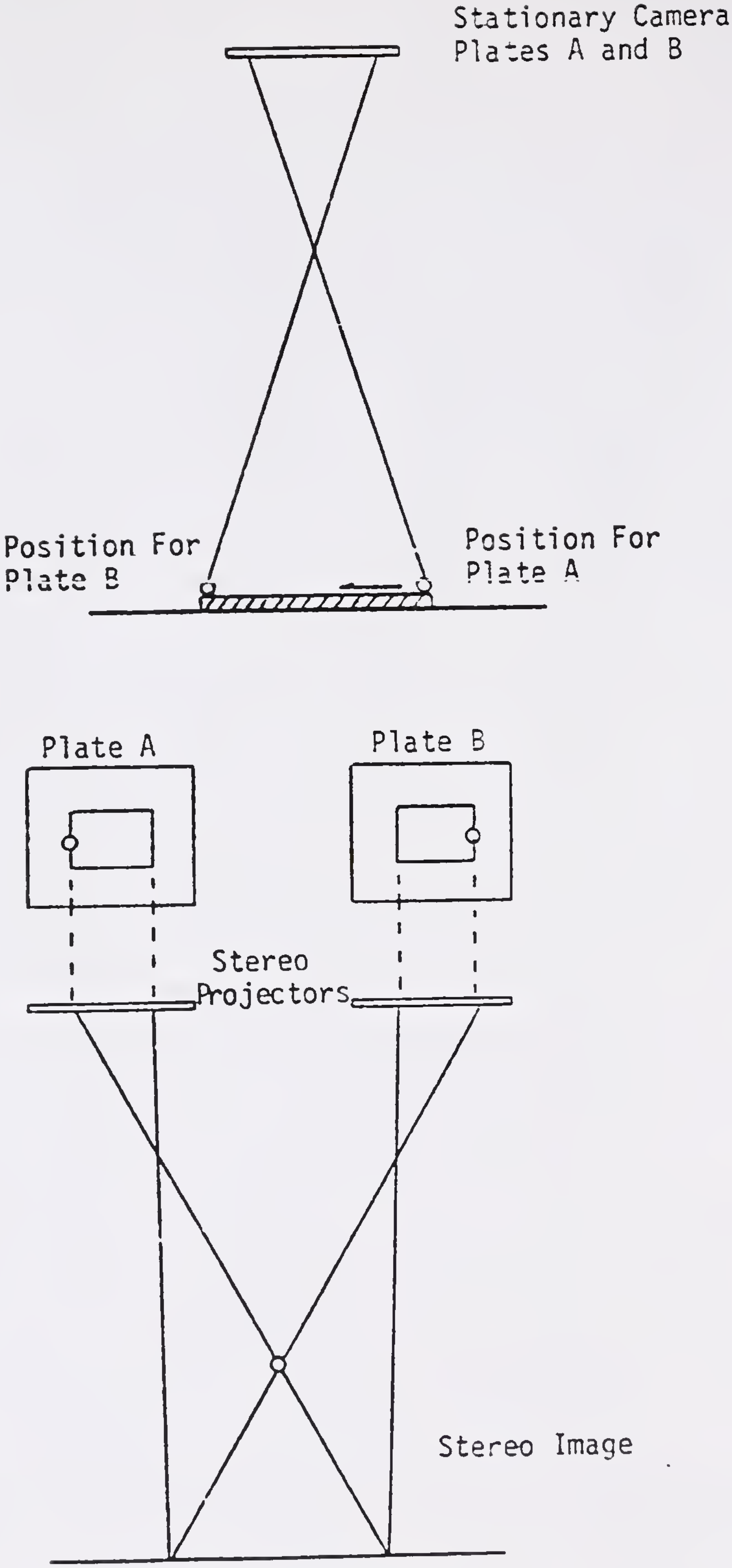


Figure 6-2. Stereo Image-Displaced Object (After Andrawes, 1976)

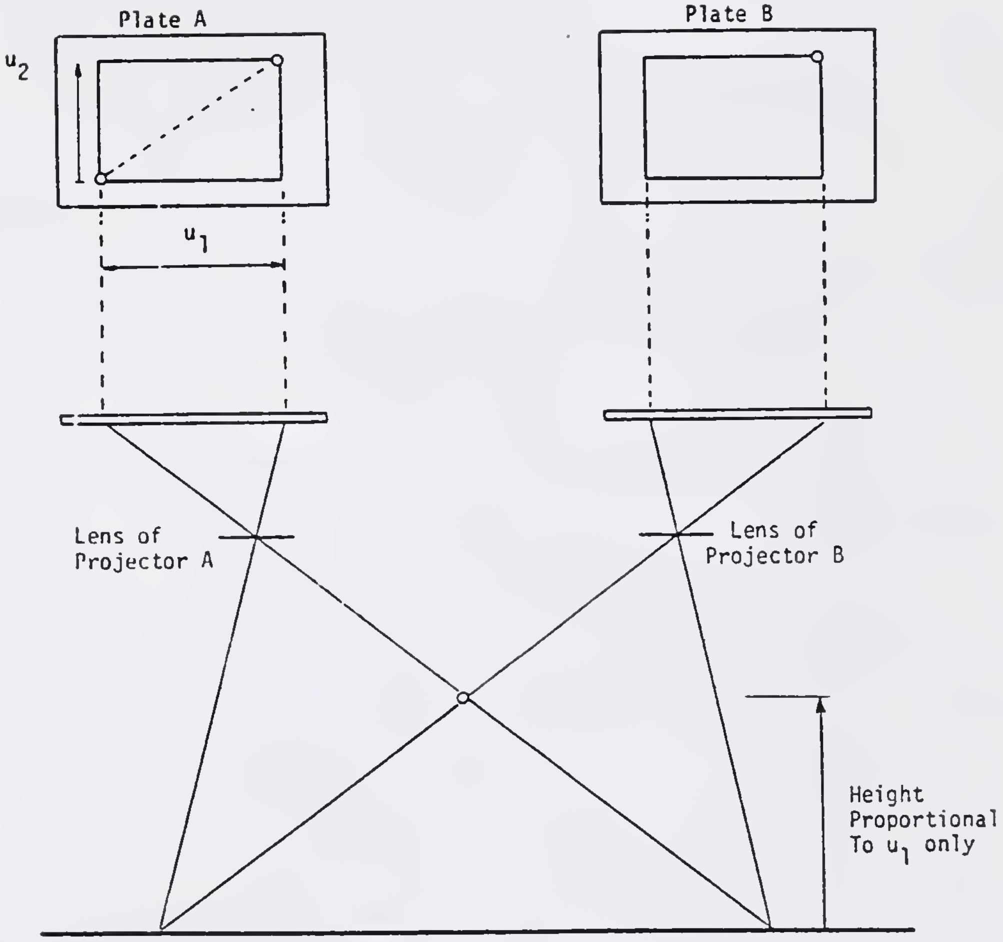


Figure 6-3. General Two-Dimensional Displacement (After Andrawes, 1976)

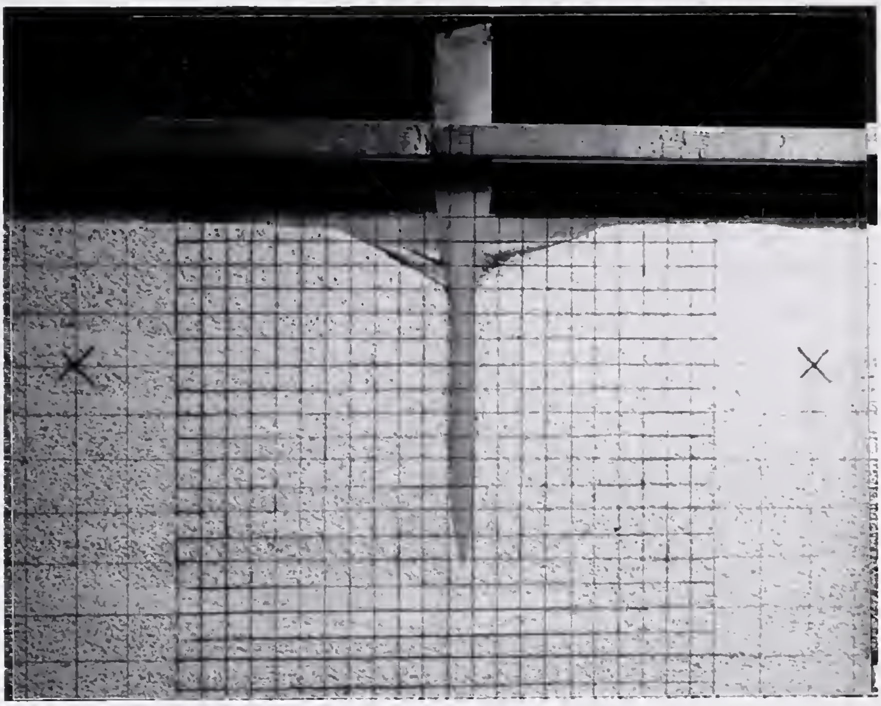


Figure 6-4. Probe Penetrated in Sand

6.2.1 Container, Probe and Sand

The test container is a wooden box with one glass wall and dimensions of 38 3/4" x 18 1/4" x 24", Figure 6-5. A grid of lines is scored on the inside face of the glass wall. The middle third length has a 15 mm x 15mm square grid while the outer two-thirds have 30 mm x 30 mm grids.

A solid dummy blade was machined with dimensions identical to those of the dilatometer and piezoblade. It was then cut longitudinally in half to provide the probe required for the stereo photography tests. A photograph is included as Figure 6-6. During a test the probe is pushed into the sand with the flat cut surface against the glass wall, by means of a hydraulic jack. Care must be taken to ensure an exactly vertical insertion and that the probe remains in tight contact with the glass.

The sand used was a new blend of sands from the Edgar Plastic Kaolin Company. It consisted of 58% 8/20 sand, 20% 20/30 sand, 12% 30/65 sand and 10% EGS sand. An 8/20 sand, for example, is one in which 75% or more of the grains fall in the range between the number 8 and number 20 sieves. In the earlier cone research the 10% fraction was MDS70 sand. This is no longer manufactured and was replaced in the current research by the Edgar Glass Sand (EGS) which has essentially the same grain size distribution. Figures 6-7 to 6-10 are photographs of the various sand components. The uniformity coefficient

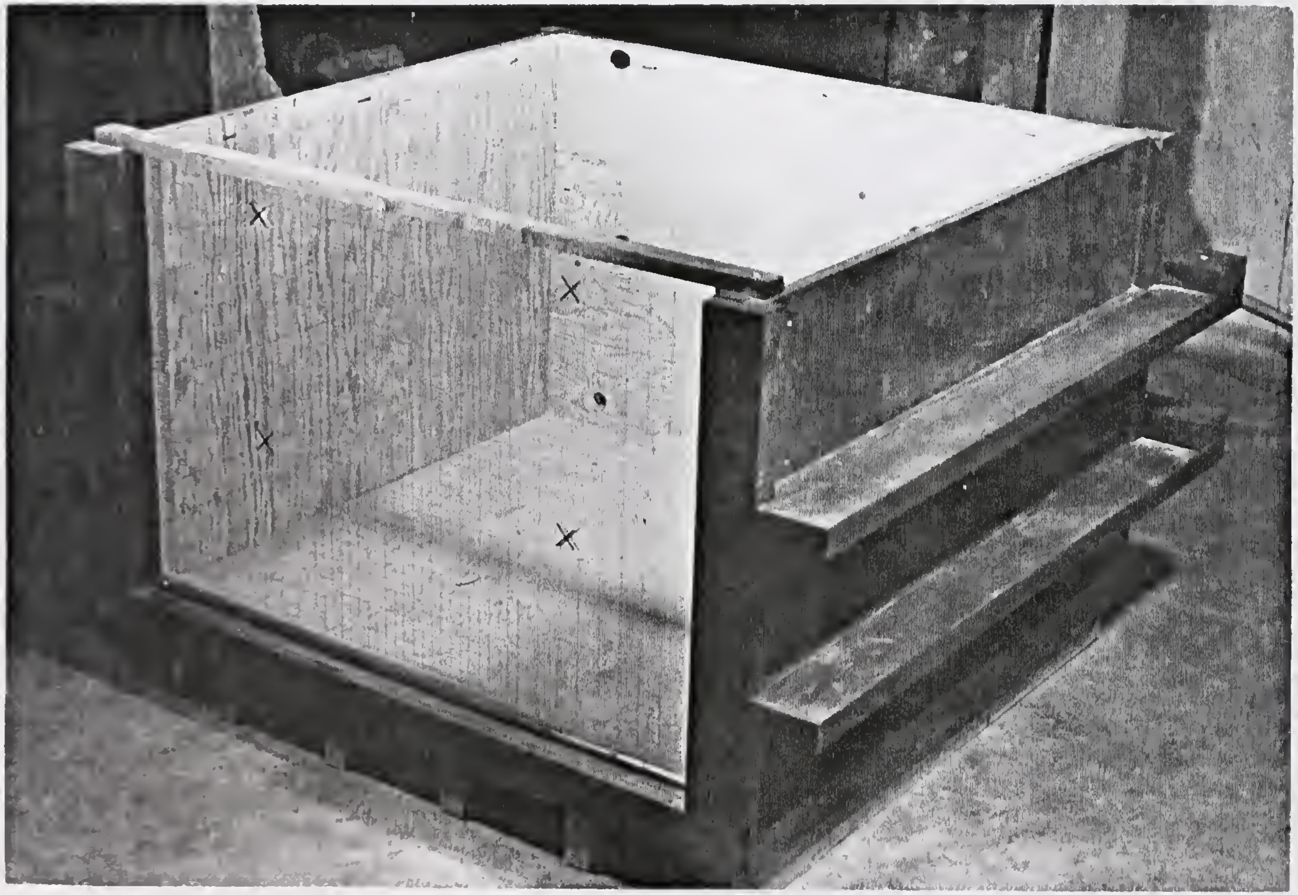


Figure 6-5. The Test Container

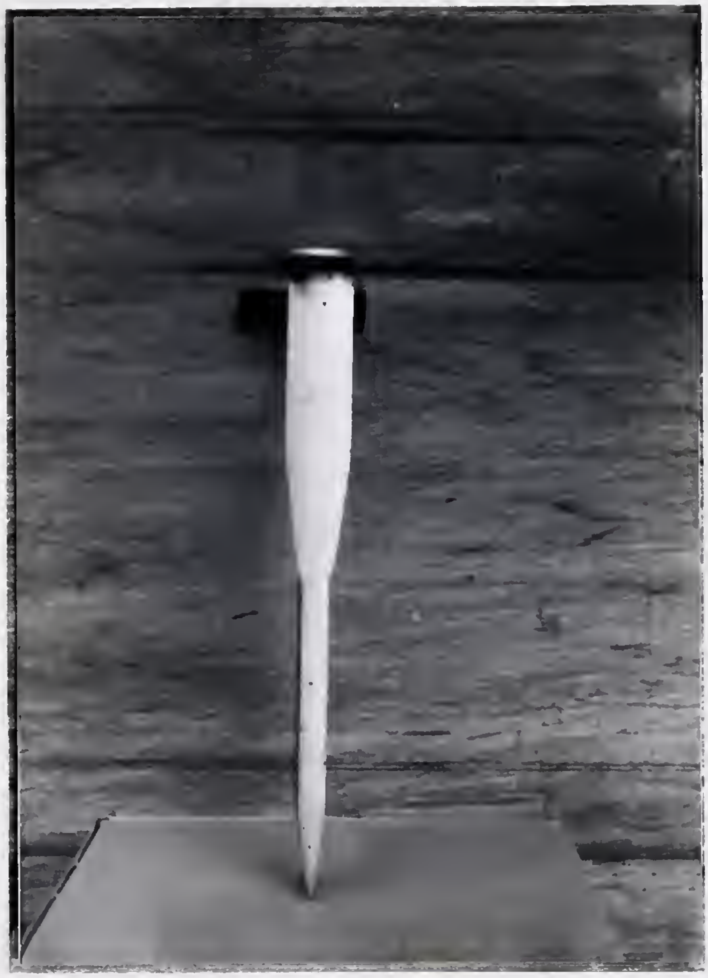


Figure 6-6. Stainless Steel Dilatometer Probe

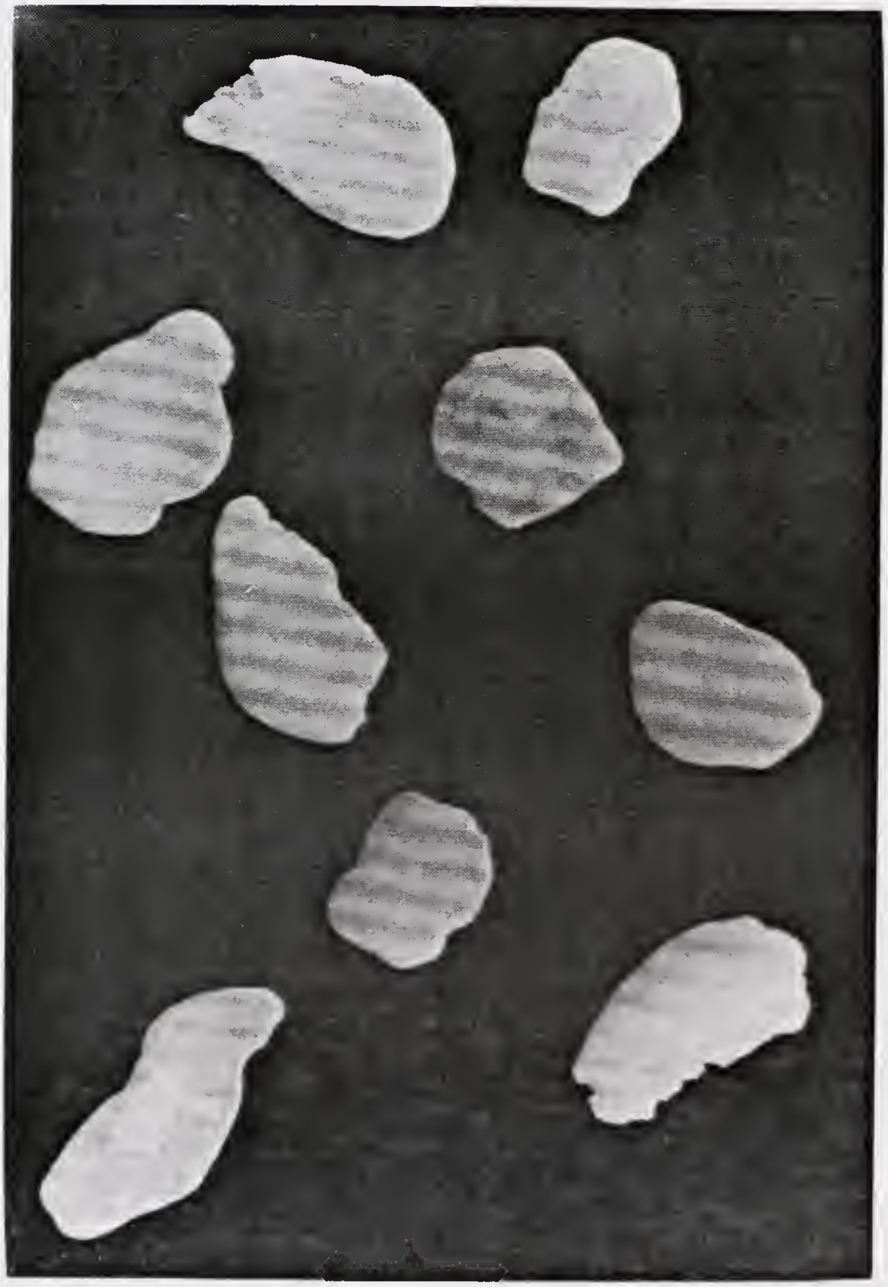


Figure 6-7. 8/20 Edgar Sand

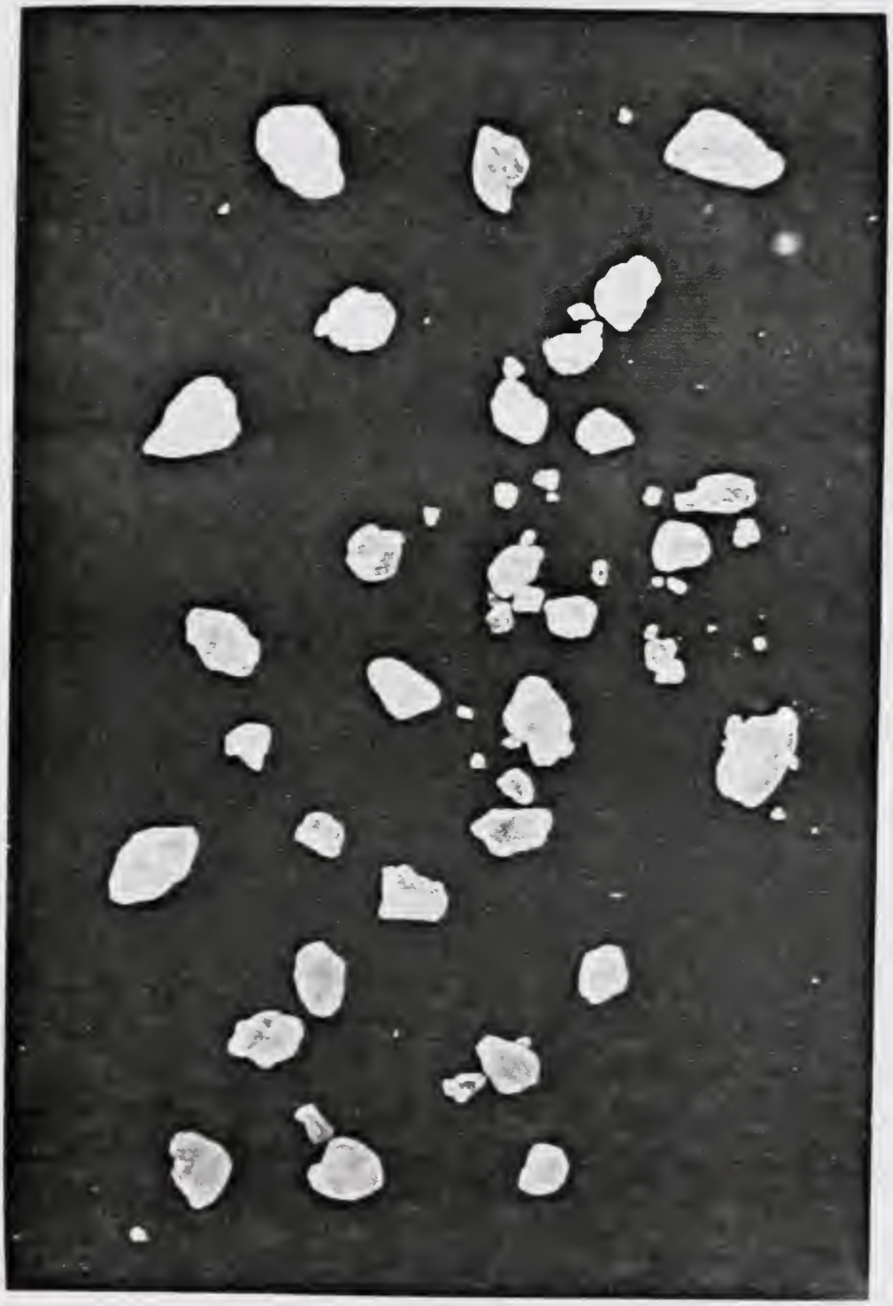


Figure 6-8. 20/30 Edgar Sand

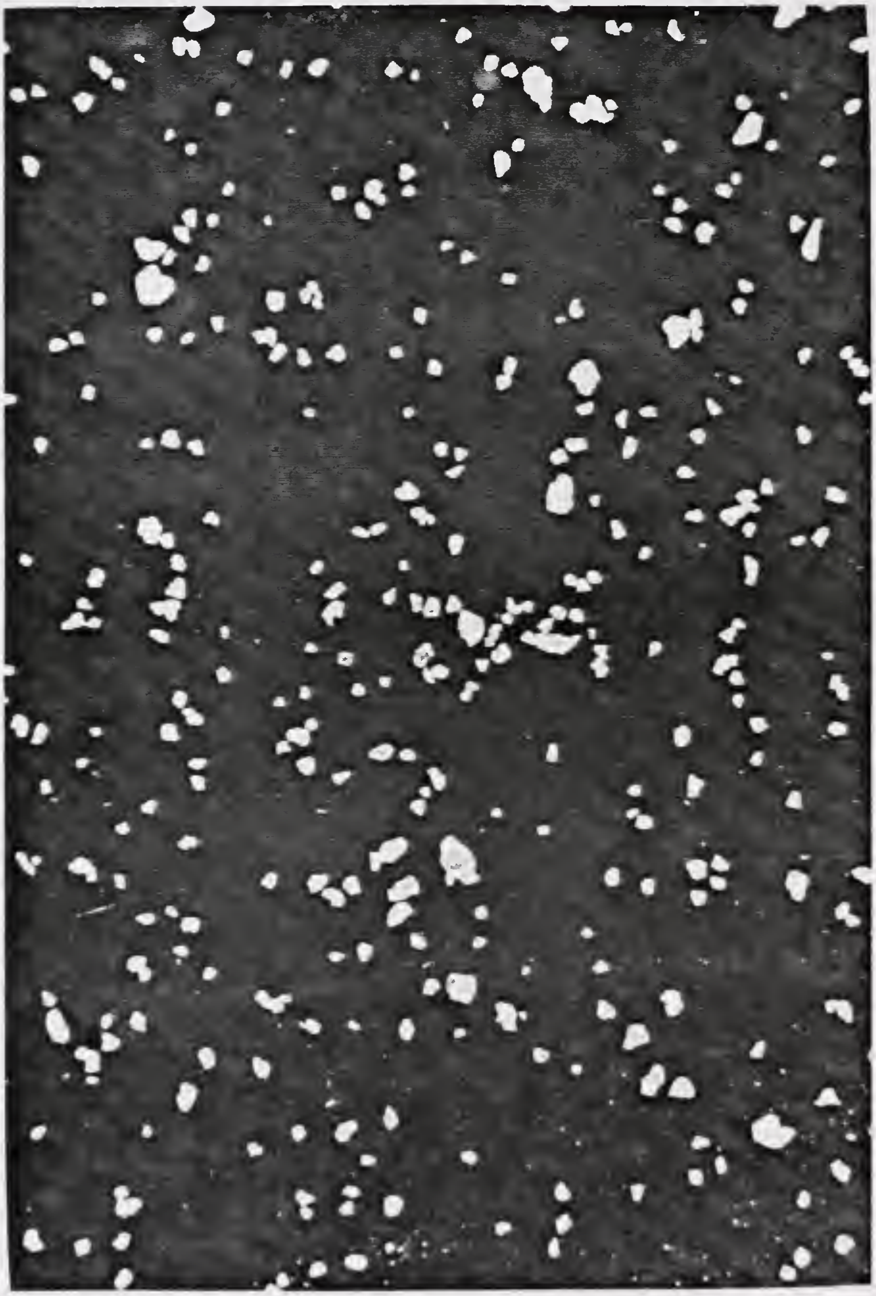


Figure 6-9. 30/65 Edgar Sand

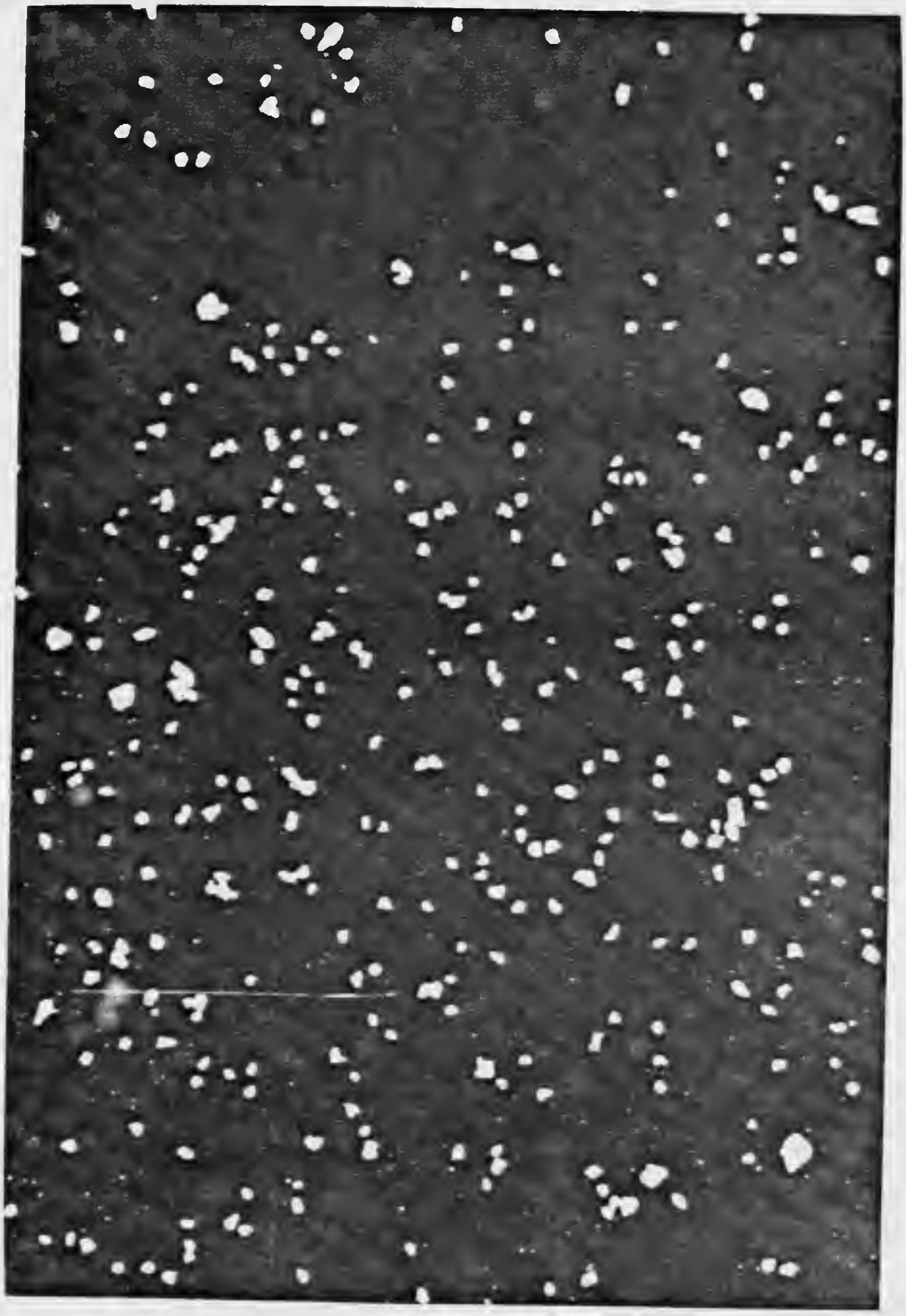


Figure 6-10. Edgar Glass Sand

of the current blend is 6.36 as compared to 6.66 for the earlier mixture. Grain size distribution curves of the components and the blend are included in Appendix F, pages 289 to 293.

6.2.2 Camera and Film

The camera used in this research was a B & J Orbit view camera with a standard back which takes 4" x 3" plate holders. The lens was an Ilex Paragon 5.5", f22-32 and the exposure time used was 0.5 seconds. The camera was located 90 cm from the glass wall in all tests. The test set-up is shown in Figure 6-11.

The photographic plates used were composed of Ektachrome 50 Professional Film (Tungsten) with a 32 ASA emulsion speed.

6.2.3 Stereo Plotter

For analysis of each pair of photographs a Fotocatigrafo Nistri Model VI projection anaglyphic plotter was used, Figure 6-12. This employs the anaglyph principle of projecting light of two complimentary colors, red and blue-green, to create the stereo image. The two photographs are projected onto a white platen. This is viewed through glasses with one red and one blue-green lens. Each eye sees only one photograph with the result that a stereo, three dimensional, image is percieved.

Since the grid scored on the glass does not move, it is used as a datum plane. Movement of the sand adjacent

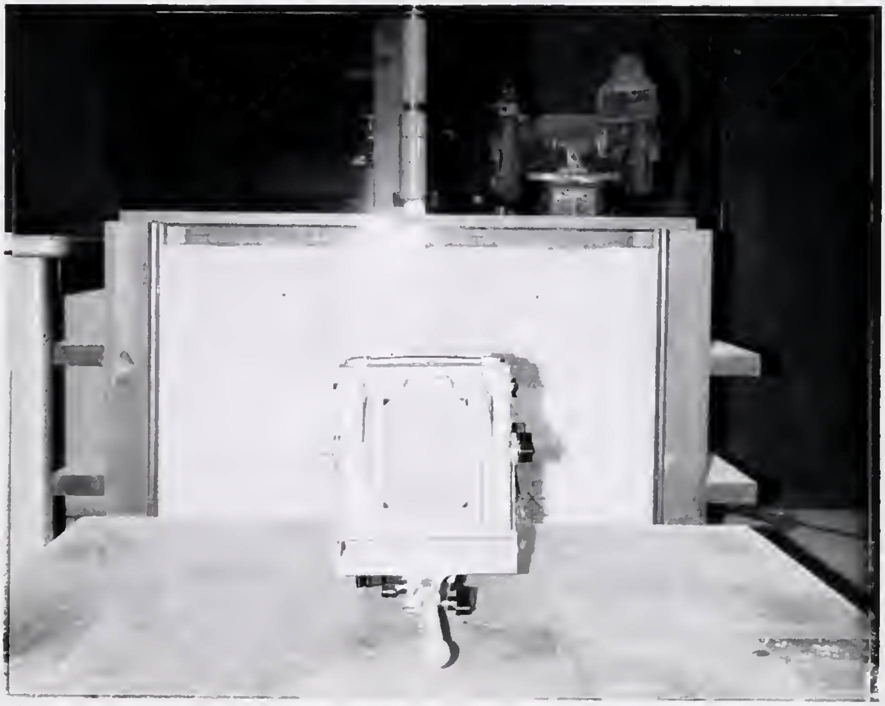


Figure 6-11. Test Set Up

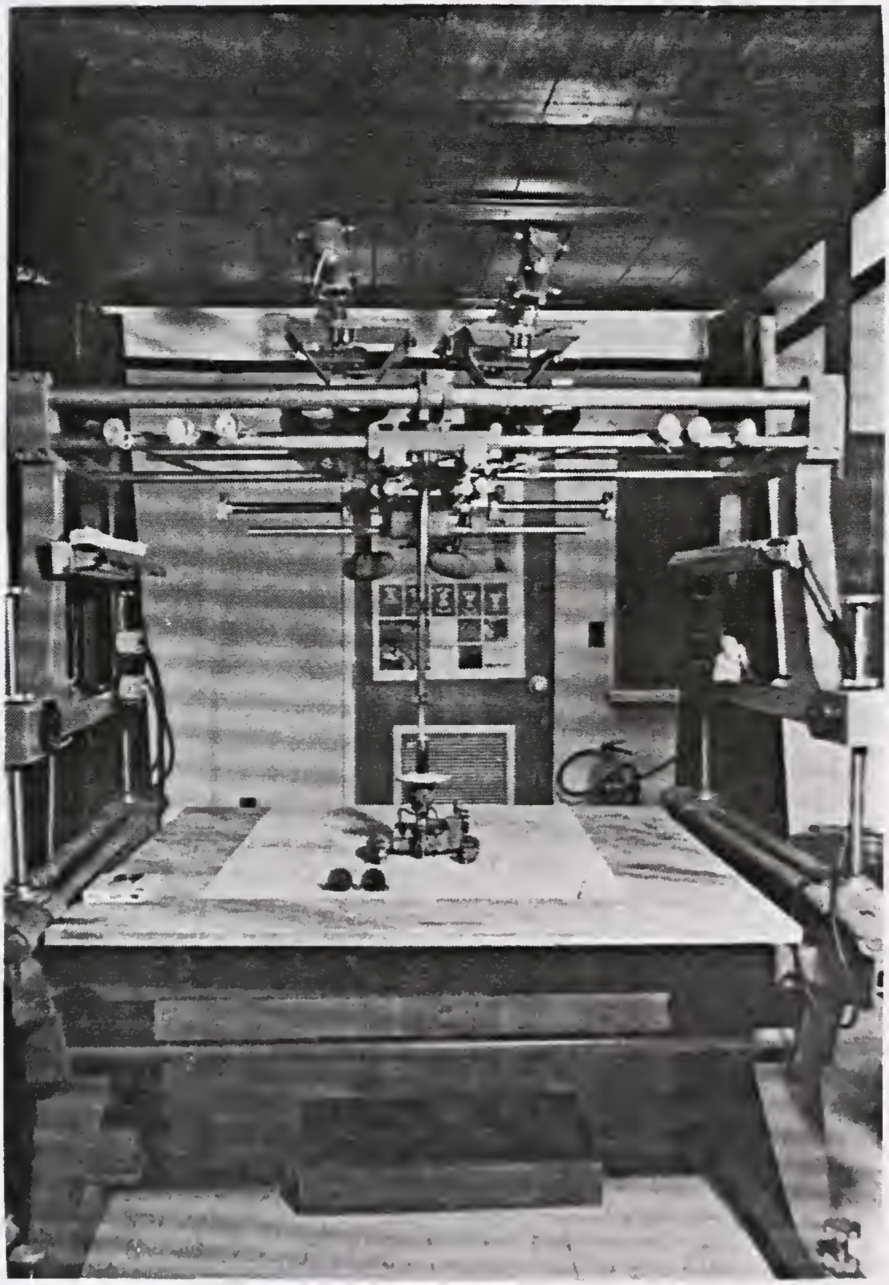


Figure 6-12. Double Projector Anaglyphic Plotter

to the glass results in a stereo image of hills and/or valleys relative to the grid. The floating dot method is used to measure image heights. The dot, which appears to be floating in space, is moved vertically by raising or lowering the platen, Figure 6-13. First, the dot is placed on a grid node point, then on the sand at the node point which may appear above or below the grid. It is usually easier to place the dot on the sand surface in a valley, so hills were usually changed into valleys by switching the lenses in the stereo glasses. The difference in the two readings taken is the image height, which is related to the sand movement at that node point in the direction joining the projectors. As noted earlier, photographs are rotated 90° to achieve the normal component of movement.

6.3 Testing Procedures

The procedures followed in the current research for sand placement, blade penetration and photograph interpretation were identical to those of the earlier cone studies and are only briefly described in this dissertation.

6.3.1 Sand Placement

Tests were performed at five different densities obtained by different levels and types of mechanical compaction. They were designated very loose, loose, medium, dense and very dense. The procedures used to fill the container were

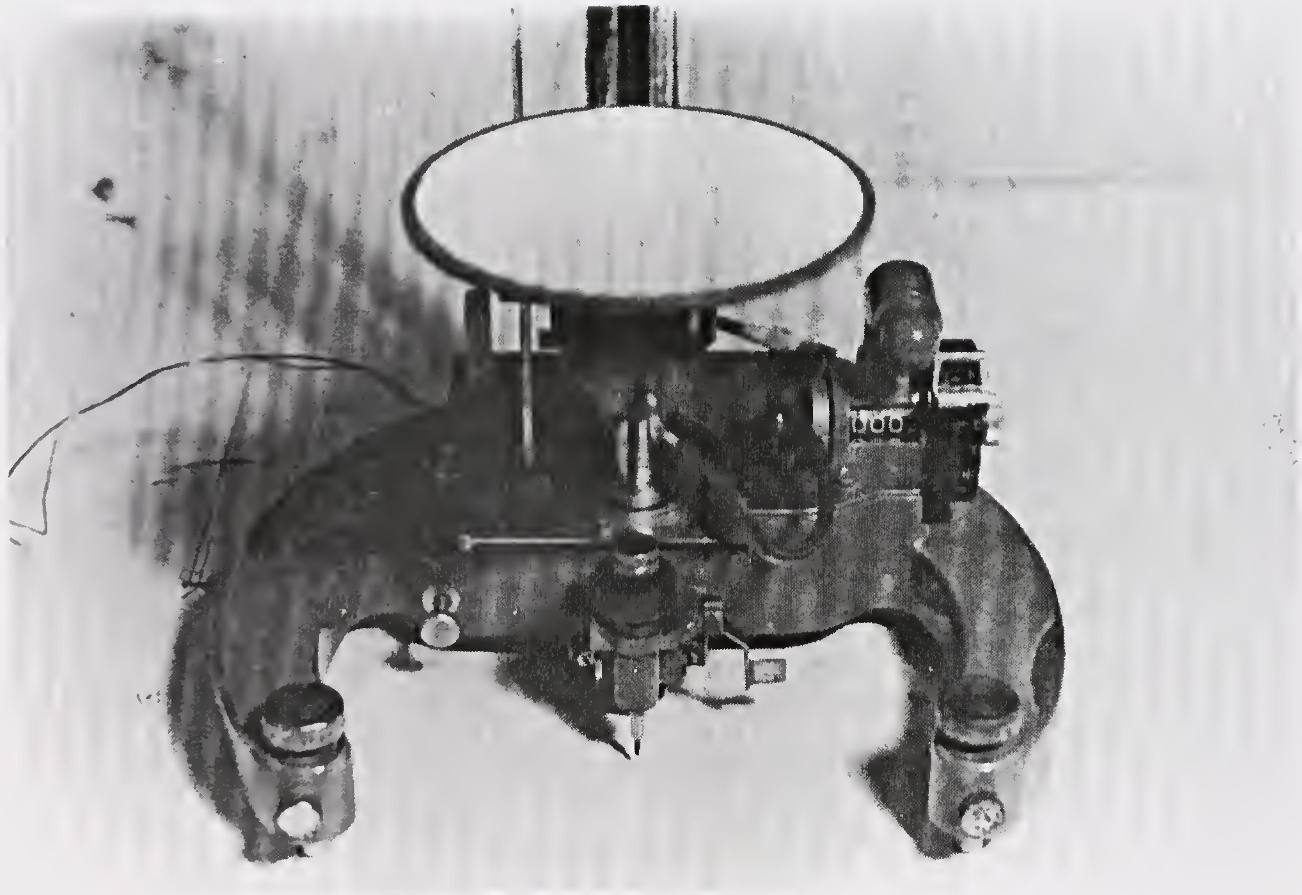


Figure 6-13. Tracer Point-Platen Assembly

Test 1: Very Loose (density 107.7 pcf)

Sand placed carefully and slowly using a 3/4" funnel held 1 to 2 inches above the surface. Total weight of sand was 1031 pounds.

Test 2: Loose (density 110.3 pcf)

Sand placed quickly using a 3/4" funnel held 5 to 7 inches above the surface. Total weight of sand was 1056 pounds.

Test 3: Medium (density 115.4 pcf)

Sand placed quickly in four inch lifts using a 3/4" funnel held 5 to 7 inches above the surface. Each lift was compacted with one pass of a mechanical compactor, Figure 6-14. Total weight of sand was 1105 pounds.

Test 4: Dense (density 118.6 pcf)

Sand placed quickly in four inch lifts using a 3/4" funnel held 5 to 7 inches above the surface. Each lift was compacted with five passes of the mechanical compactor. Total weight of sand was 1135 pounds.

Test 5: Very Dense (density 126.4 pcf)

Sand placed quickly in four inch lifts using a 3/4" funnel held 5 to 7 inches above the surface. Each lift was compacted with two passes of a small vibrating compactor, Figure 6-14, held at each position for 10 seconds and one pass of the mechanical compactor. Total weight of sand was 1210 pounds.

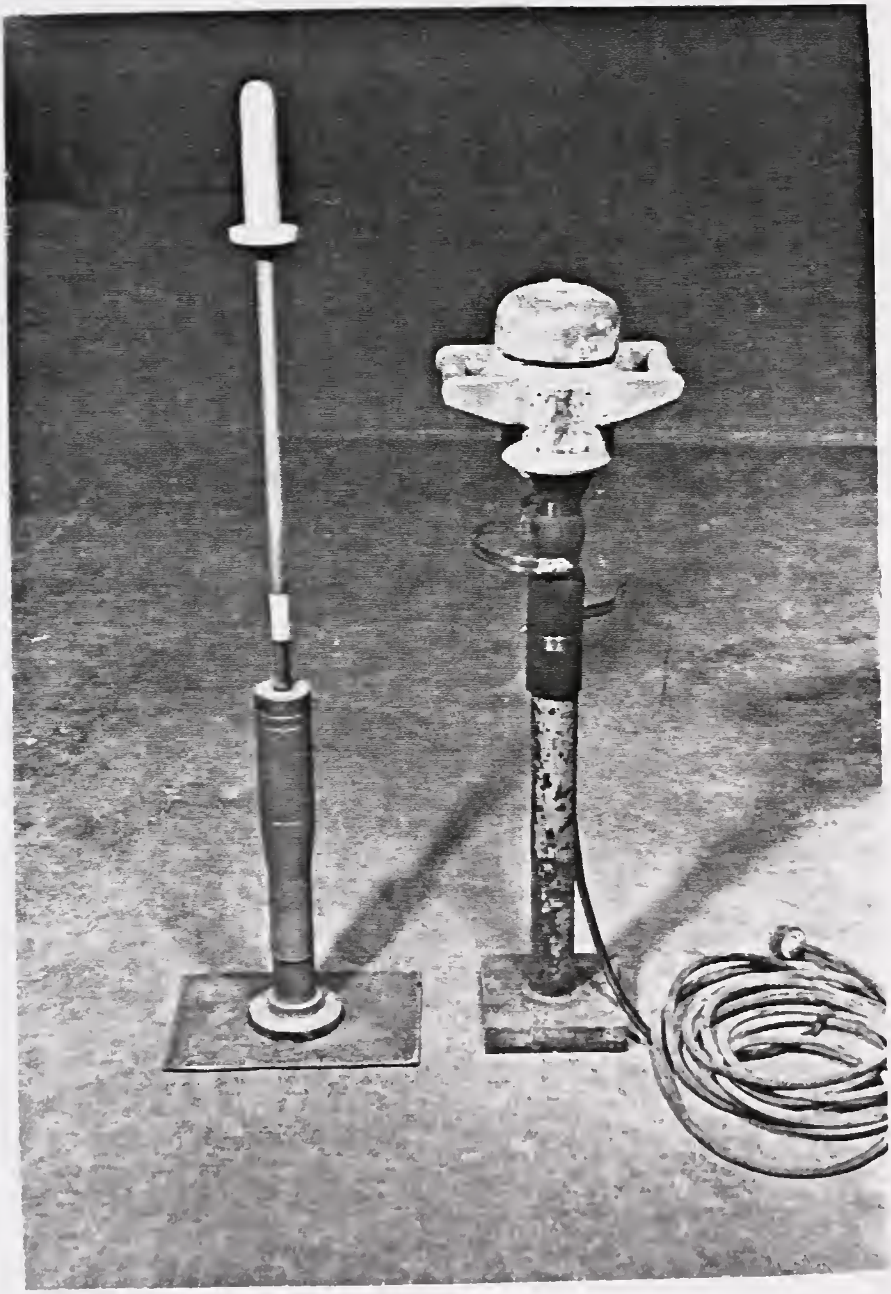


Figure 6-14. Compaction Equipment. Left-Vibratory Compactor. Right-Constant Energy.

6.3.2 Blade Penetration

A photograph of the test set up was included as Figure 6-11. The camera was fixed to a weighted table and in all tests was located in exactly the same position. The first test photograph was taken of the undisturbed sand prior to blade insertion. The blade was then penetrated into the sand, adjacent to the glass wall, using a small hydraulic jack, Figure 6-15. Further photographs were taken at different penetration depths, e.g., Figure 6-4. Two spotlights were used to brightly illuminate the sand.

6.3.3 Parallax Measurements

The first (undisturbed sand) and last (final penetration) photographs for each test were viewed in the stereo plotter to determine the sand grain movement around the penetrated blade. Copies of these 10 photographs for the 5 tests are included in Appendix F, pages 294 to 298 . The grid scored on the glass wall was numbered as shown in Figures 6-16 and 6-17 for the purpose of data recording and computer reduction. There were 1156 elements and 1234 node points, i.e., intersection or corner points. Readings were not made at all 1234 points but only on one side of the symmetrical probe and to an extent where movements were discernible.

Photographs were first viewed to determine the horizontal components of movement, called x-parallax.

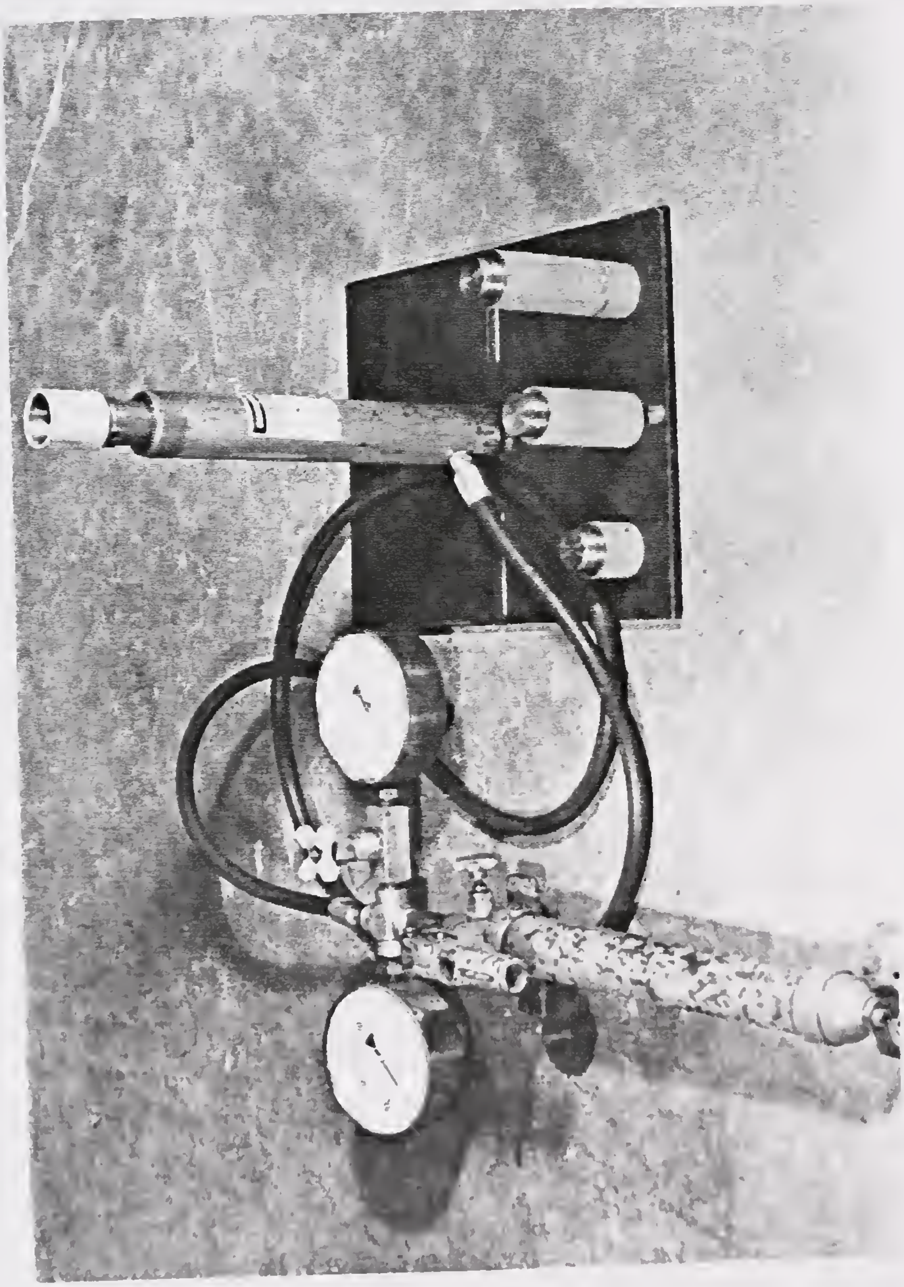


Figure 6-15. Jack and Extensions

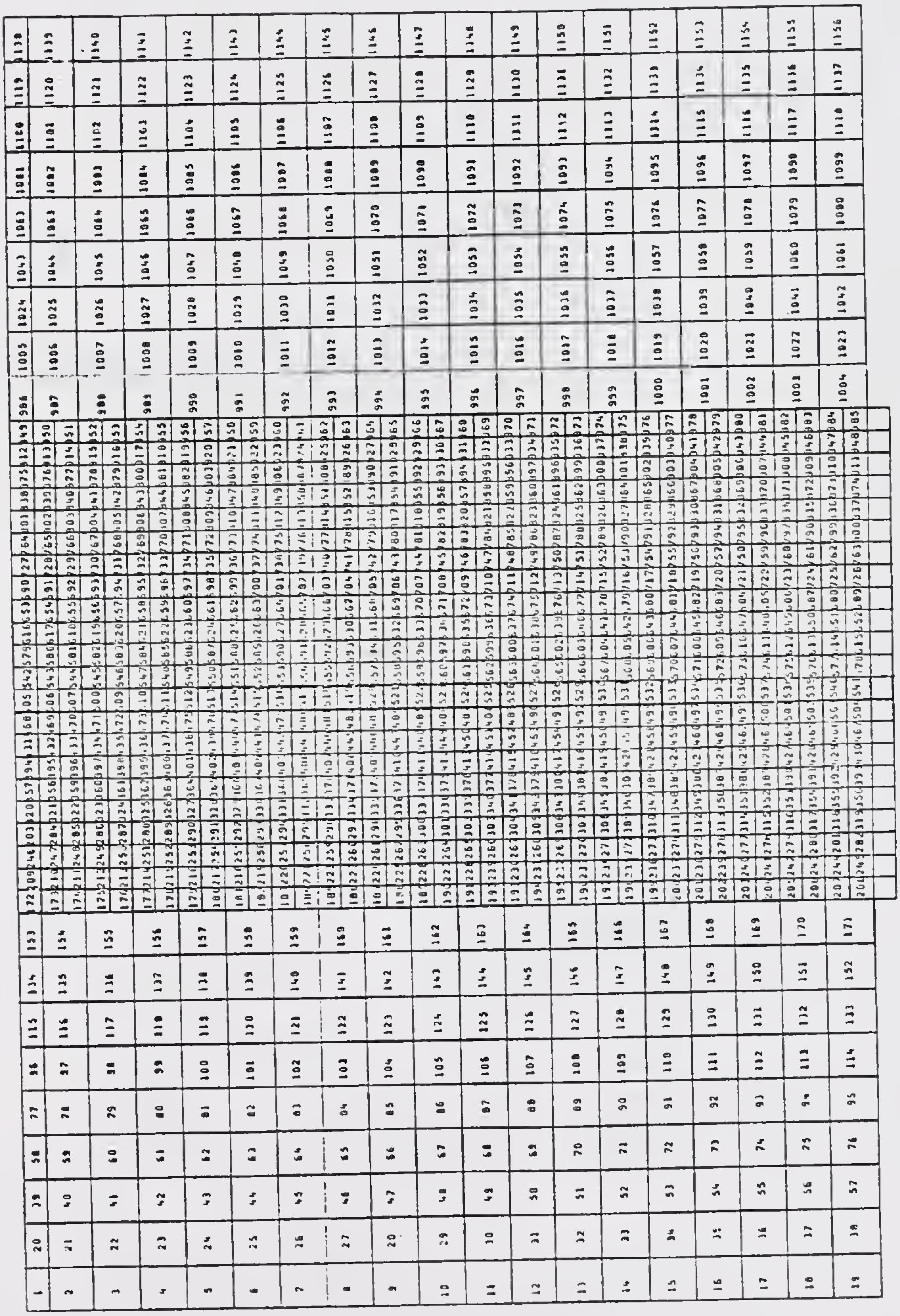


Figure 6-16. Grid Element Numbering System (After Barreiro, 1978)

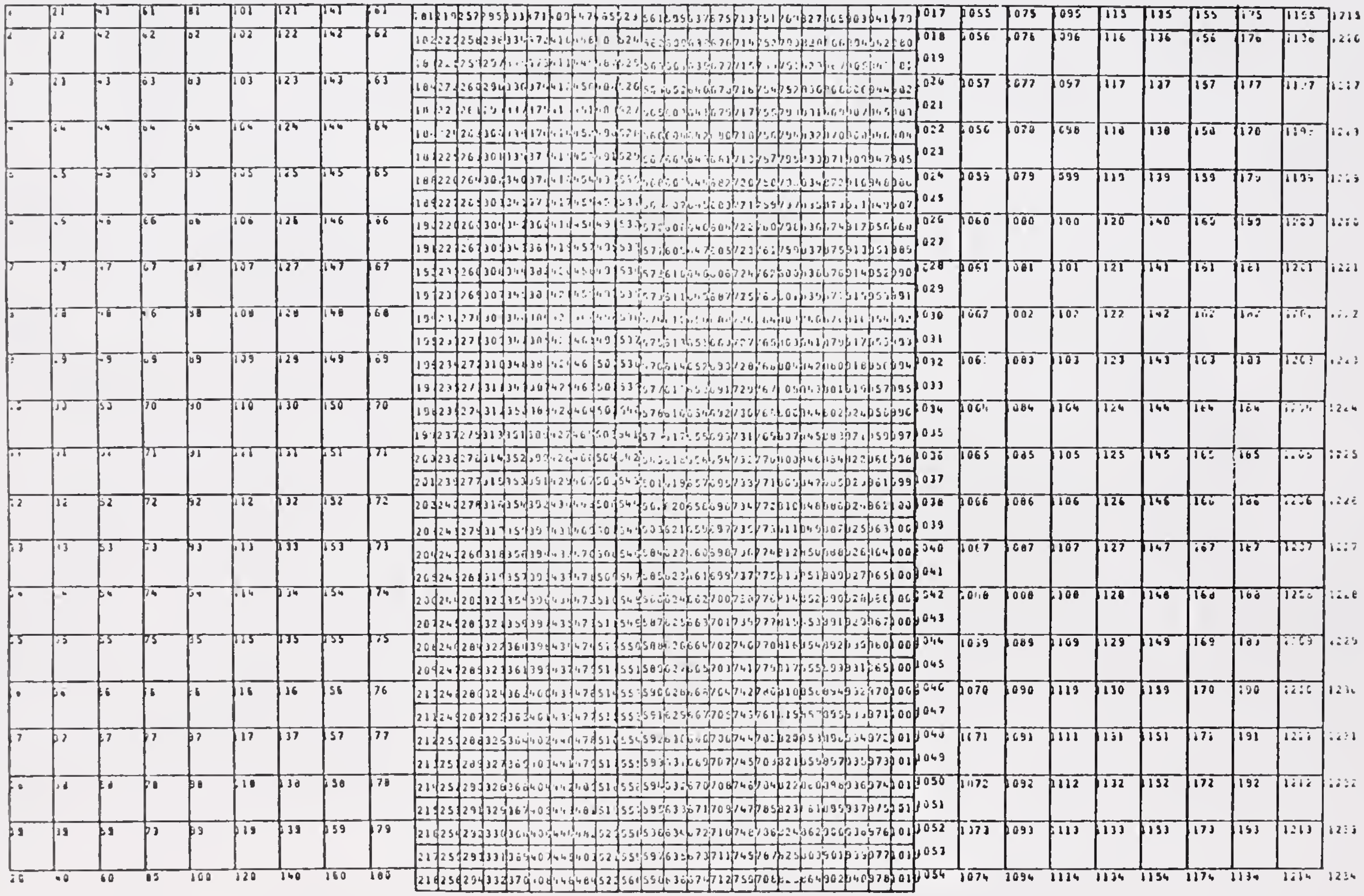


Figure 6-17. Node Point Numbering System (After Barreiro, 1978)

They were then rotated 90° and the y-parallax viewed to obtain the vertical movement components. Two readings were taken at each node point. First, the floating dot was placed on the grid, then on the apparent sand surface. If, when viewing x-parallax, there is at the point considerable y-parallax, it may be impossible to obtain the second reading. Mortensen (1979) introduced the technique of moving the blue-green projector in the y-direction to eliminate the y-parallax. This does not affect the x-parallax reading.

6.4 Scale Determination, Displacements and Strains

6.4.1 Scales

Procedures for determining the relationships between camera-model and projector-stereo image were defined by Barreiro (1978). The final equation relates parallax measurement to actual displacement in millimeters. The variables used in the analysis are defined below and in Figure 6-18.

ΔX models actual motion

$\Delta X'$ motion recorded on film

$\Delta X''$ apparent projected motion

Y any known distance in the model

Y' this known distance as it appears in the projected image

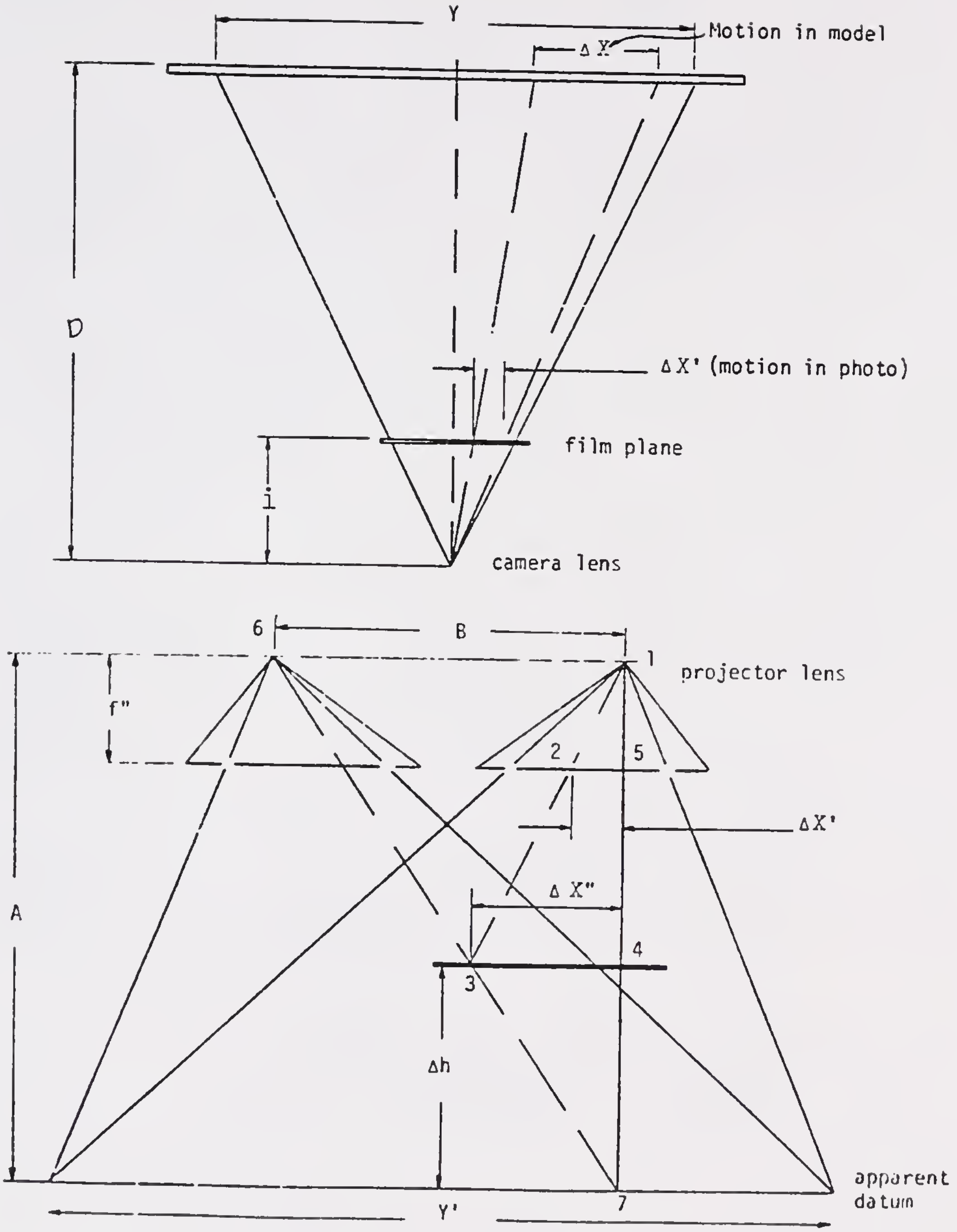


Figure 6-18. Camera - Model and Projector-Stereo Image Relationships (After Davidson, 1980a)

i	image length of the camera lens (assumed equal to the focal length of the camera lens)
D	distance from camera lens to the model
f"	focal length of projector lens
A	distance from projector lens to apparent datum, the table
B	distance separating the two projectors
h	apparent height observed in stereo image, created by the displacement on photographic slides.

The assumption that the image length (i) is equal to focal length (f) allows the motion of the particles in the photographic slides to be defined.

$$\Delta X' = \Delta X(f/D) \tag{6-1}$$

By similar triangles (1-2-5 and 1-3-4) in Figure 6-18 the apparent projected motion

$$\Delta X'' = \Delta X'(A - \Delta h)/f'' \tag{6-2}$$

Also using similar triangles (6-7-1 and 3-4-7)

$$\frac{\Delta h}{A} = \frac{\Delta X''}{B} = \left(\frac{\Delta X'(A - \Delta h)/f''}{B}\right) \tag{6-3}$$

Substituting equation (6-1) into equation (6-3)

$$\frac{\Delta h}{A} = \frac{\Delta X(f/D)(A - \Delta h)}{f''B} \tag{6-4}$$

Solving for ΔX, the actual motion in the model

$$\Delta X = \frac{D \cdot f'' \cdot B \cdot \Delta h}{A \cdot f \cdot (A - \Delta h)} \qquad (6\text{-}5)$$

The projected distance Y' can be expressed in terms of the actual distance Y

$$Y' = Y(f/D)(A/f'') \qquad (6\text{-}6)$$

A scale ratio (K) can be defined as

$$K = (Y'/Y) = (f/D)(A/f'') \qquad (6\text{-}7)$$

Solving for ΔX in equation (6-5)

$$\Delta X = (D/f)(f''/A)\left(\frac{B \cdot \Delta h}{A - \Delta h}\right) \qquad (6\text{-}5a)$$

Putting K from equation (6-7) into equation (6-5a)

$$\Delta X = (1/K)\left(\frac{B \cdot \Delta h}{A - \Delta h}\right) \qquad (6\text{-}8)$$

The platen tracer dial assembly can accomodate various gears with different gear ratios. In this research, gears were chosen to provide a 1:750 ratio, giving each division on the dial assembly equivalent to 0.1 foot of real elevation. Below are the equations utilized to calculate Δh.

$$\Delta h(\text{mm}) = \left(\frac{\text{reading 1-reading 2}}{750\ \text{ft./ft.}}\right)(12\ \text{in./ft.})(25.4\ \text{mm/in.}) \qquad (6\text{-}9)$$

$$\Delta h(\text{mm}) = \text{reading}\ (0.04064) \qquad (6\text{-}10)$$

In the denominator of equation (6-8), Δh was assumed negligible compared to the measured distance A. Thus equation (6-8) becomes

$$\Delta X = (1/K)(B/A)\Delta h \qquad (6\text{-}11)$$

The following distances were measured:

$$A = 89.90 \text{ cm}$$

$$B = 33.50 \text{ cm}$$

$$Y = 45.38 \text{ cm}$$

$$Y' = 41.90 \text{ cm}$$

Solving for K from equation (6-7)

$$K = (Y'/Y) = \frac{41.90}{45.38} = 0.923314$$

Putting the values for K, A, B, and Δh into equation (6-11)

$$\Delta X = \frac{1}{0.923314}\left(\frac{33.50}{89.90}\right)(0.04064)\Delta\text{reading}$$

so

$$\Delta X = (.016402)\Delta\text{reading} \qquad (6\text{-}12)$$

Equation (6-12) is used to determine the actual displacement at each node point in the model in millimeters, given the difference in readings taken from the stereo image.

6.4.2 Displacements

The actual horizontal and vertical displacements at each node point were calculated using equation (6-12). The resultant displacement D was then found using the equation

$$D = (U^2 + W^2)^{\frac{1}{2}} \tag{6-13}$$

where

U = horizontal displacement

W = vertical displacement

The angle which the displacement D makes with the horizontal is given by

$$\alpha = \tan^{-1}(W/U) \tag{6-14}$$

6.4.3 Volumetric Strain

The computer program developed by Barreiro (1978) for reduction of the data, computes normal strains, shear strains, volumetric strains, and density changes. In this research only the volumetric strains were used.

$$\varepsilon_v = \frac{\Delta V}{V} \tag{6-15}$$

where

ε_v	volumetric strain
ΔV	change in volume
V	original volume

For the axisymmetrical pattern

$$\varepsilon_v = \frac{A' - A}{A} \qquad (6\text{-}16)$$

A' new deformed area

A original area

The deformed area of each element was calculated knowing the resultant displacements of the four corner node points. Volumetric strains are presented as percentages and with compression defined as positive.

6.5 Analysis of Stereo Photography

The x- and y-parallax readings from the five tests were punched onto cards, along with the corresponding node point numbers, and used as input for the computer reduction program. The outputs of interest were the displacement magnitudes (D) and directions (α) of the sand movement at each node point and the volumetric strains (ε_v) for each element. Some typical computer output is included in Appendix F. The displacements at each node point were plotted to scale and contours of volumetric strain drawn, Figures 6-20 to 6-24. The corresponding five figures for the cone, from Davidson (1980a), are also included in Appendix F on pages 299 to 303.

6.5.1 Very Loose Sand (Figure 6-19)

All displacements were to the right and down away from the probe. The entire stereo image appeared as a large valley for both x- and y-parallaxes. The effects of displacement were measurable laterally to a distance

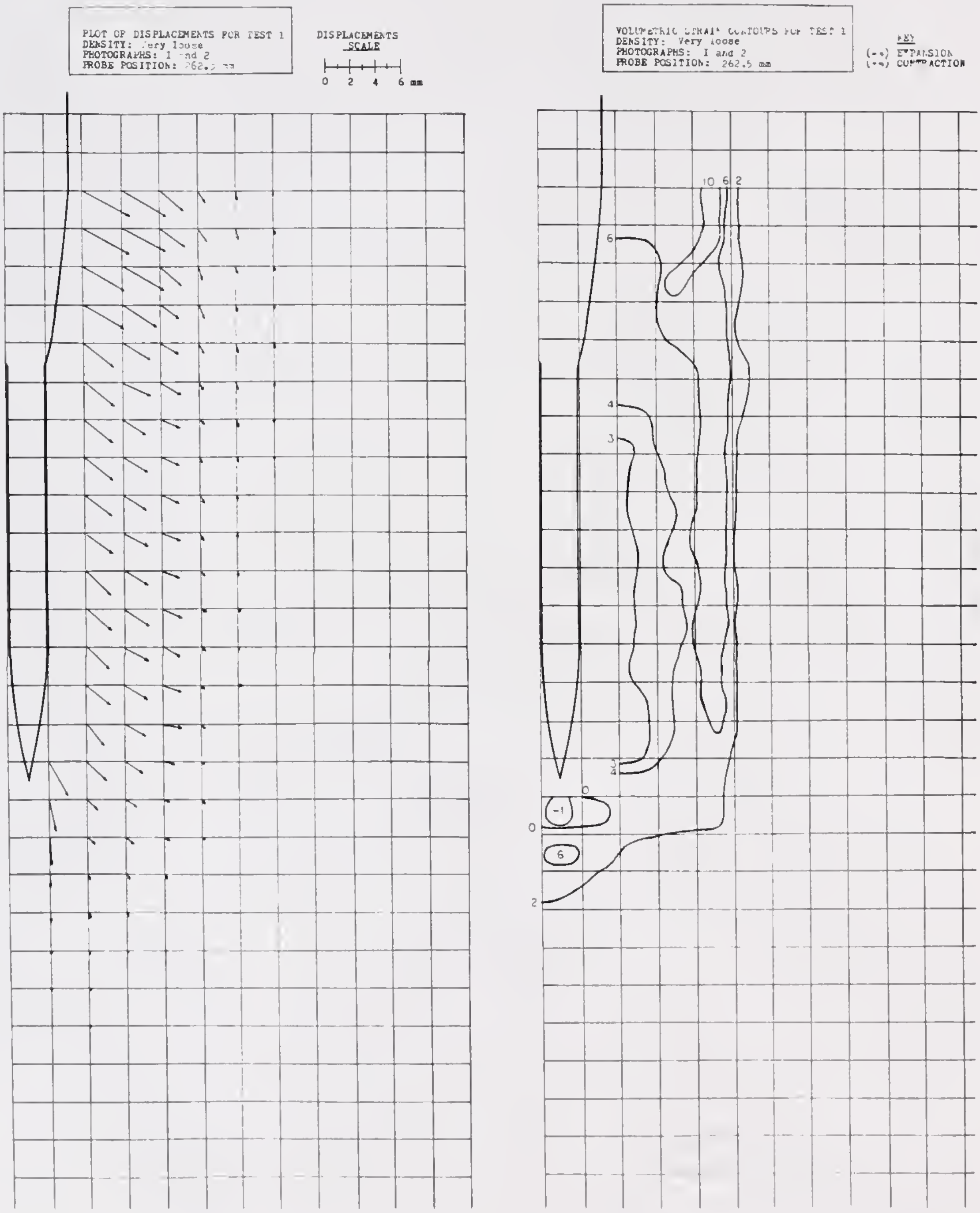


Figure 6-19. Plots of Displacement and Volumetric Strain for Very Loose Sand

of 80 millimeters from the edge of the probe and to a distance of 70 millimeters from the tip of the probe. The maximum displacement in the flat area of the probe was about three millimeters.

The observed volumetric strains were all positive (compression) except at the very tip of the probe. The contours showed a quite uniform change in strain along the probe.

6.5.2 Loose Sand (Figure 6-20)

In the loose sand, the displacements were also all to the right and downward. This was evident from viewing the stereo image, which showed a large valley area. The maximum outward movements were to a distance of 100 millimeters from the edge of the probe and downward to a depth of 100 millimeters below the tip. The magnitude of maximum displacement along the flat area of the probe was 2.2 millimeters.

As in the very loose test, the volumetric strain at the tip showed some negative values (expansion). The contours of the volumetric strain were uniform and the effect of densification became smaller with distance from the probe.

6.5.3 Medium Sand (Figure 6-21)

The x-parallaxes for the medium soil were all positive, i.e., all displacement to the right. This was obvious from

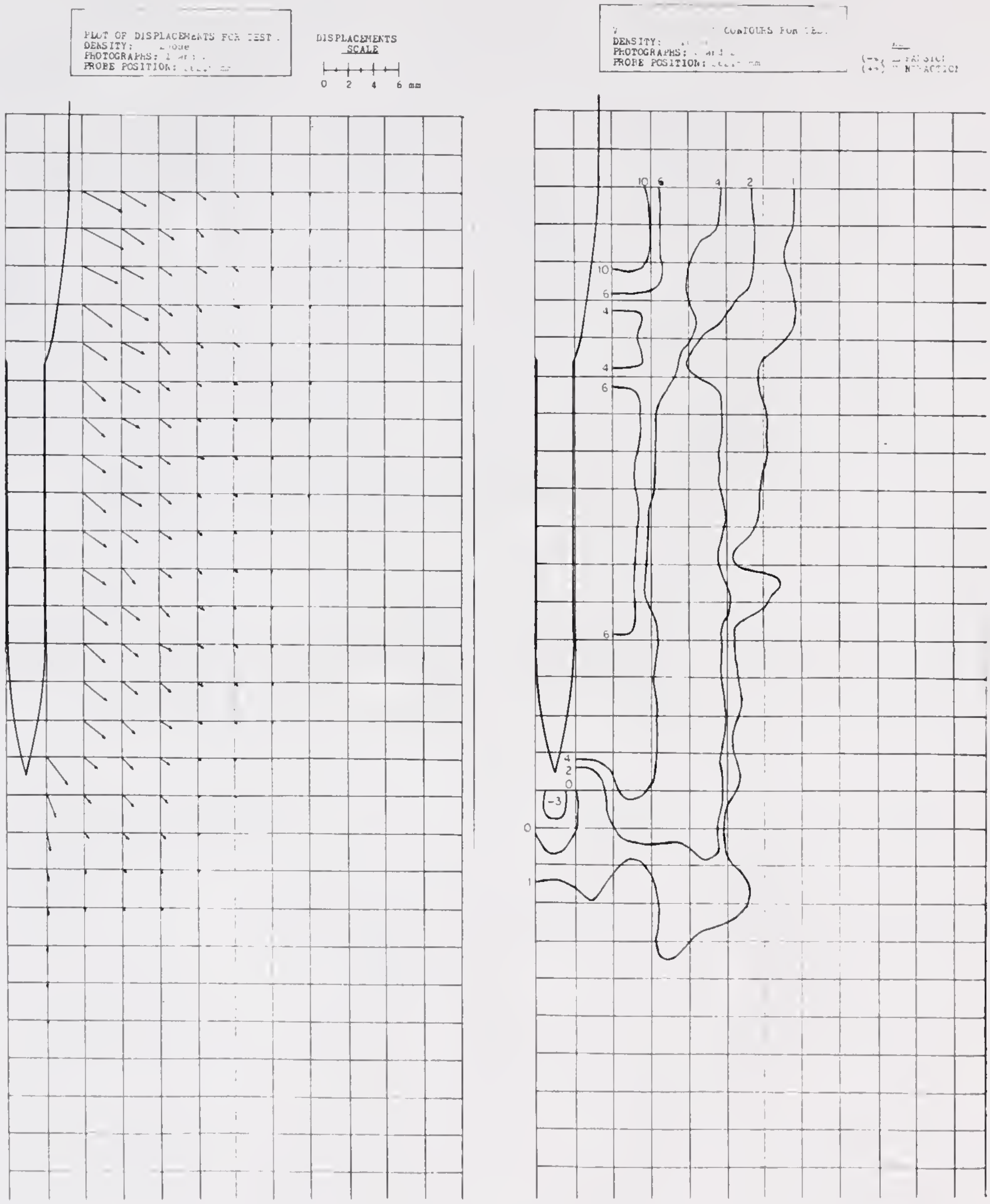


Figure 6-20. Plots of Displacement and Volumetric Strain for Loose Sand

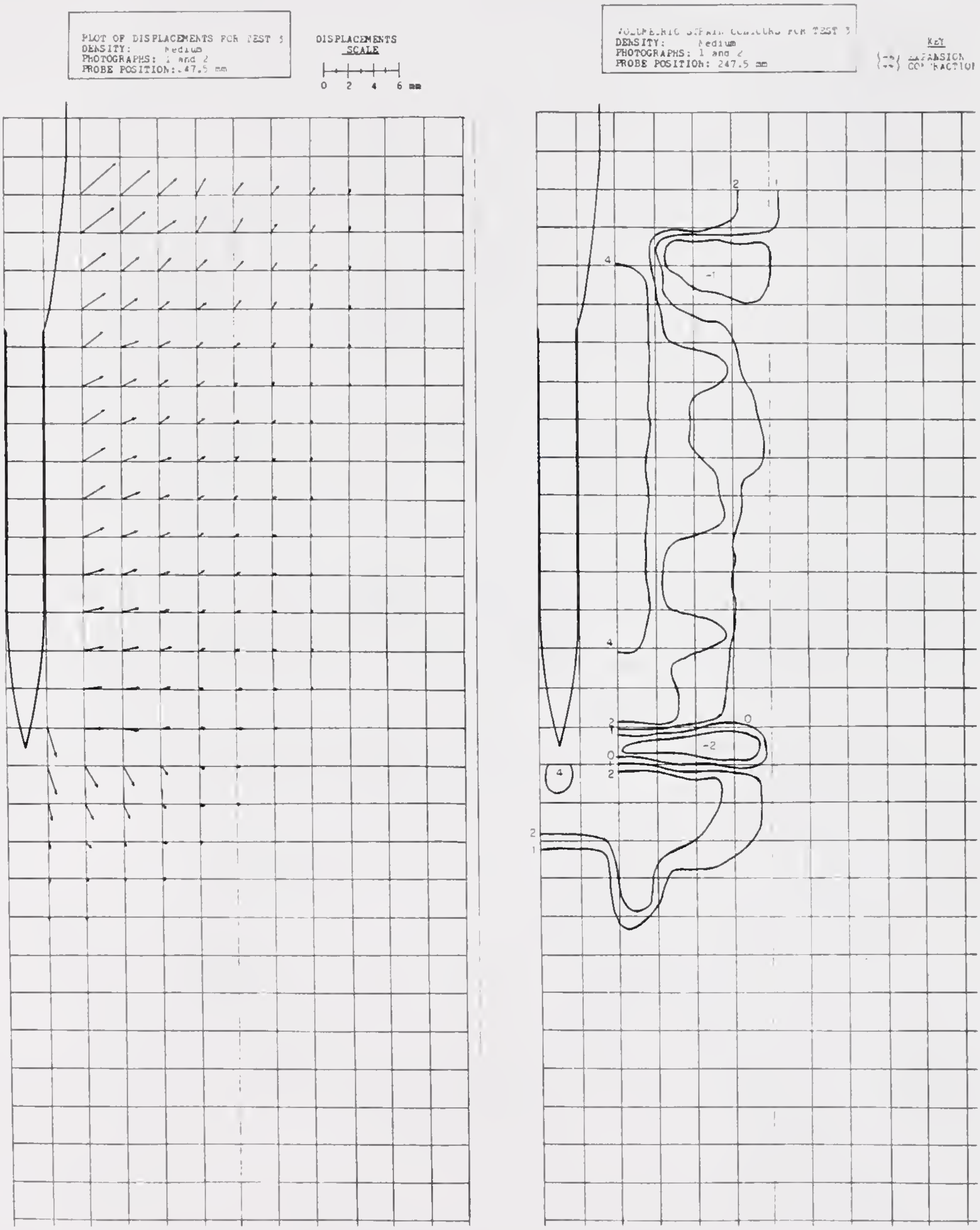


Figure 6-21. Plots of Displacement and Volumetric Strain for Medium Sand

the large valley present in the stereo image. The y-parallaxes were both positive and negative. The soil adjacent to most of the probe showed as a hilly region indicating upward movement. However, very close to the tip, valleys were viewed indicating a downward movement. The outward movements were measurable to a distance of 110 millimeters from the edge of the probe, and downward movement to a depth of 70 millimeters from the tip. The maximum displacements were of the order of two millimeters and decreased with distance from the probe.

The contours of volumetric strain showed two negative (expansion) zones, one near the top of the probe and one adjacent to the tip, where the y-parallaxes changed direction. The contours showed a fairly uniform change in volumetric strain along the flat part of the probe.

6.5.4 Dense Sand (Figure 6-22)

As in all the tests, the x-parallaxes were again positive, indicating movement to the right. The parallax reading in the y-direction again showed hills adjacent to most of the probe and valleys near the tip.

The outward movement was measurable to a distance of 160 millimeters from the edge of the probe and downward to a depth of 90 millimeters below the tip. The maximum node point movement in the flat area of the probe was 3.6 millimeters.

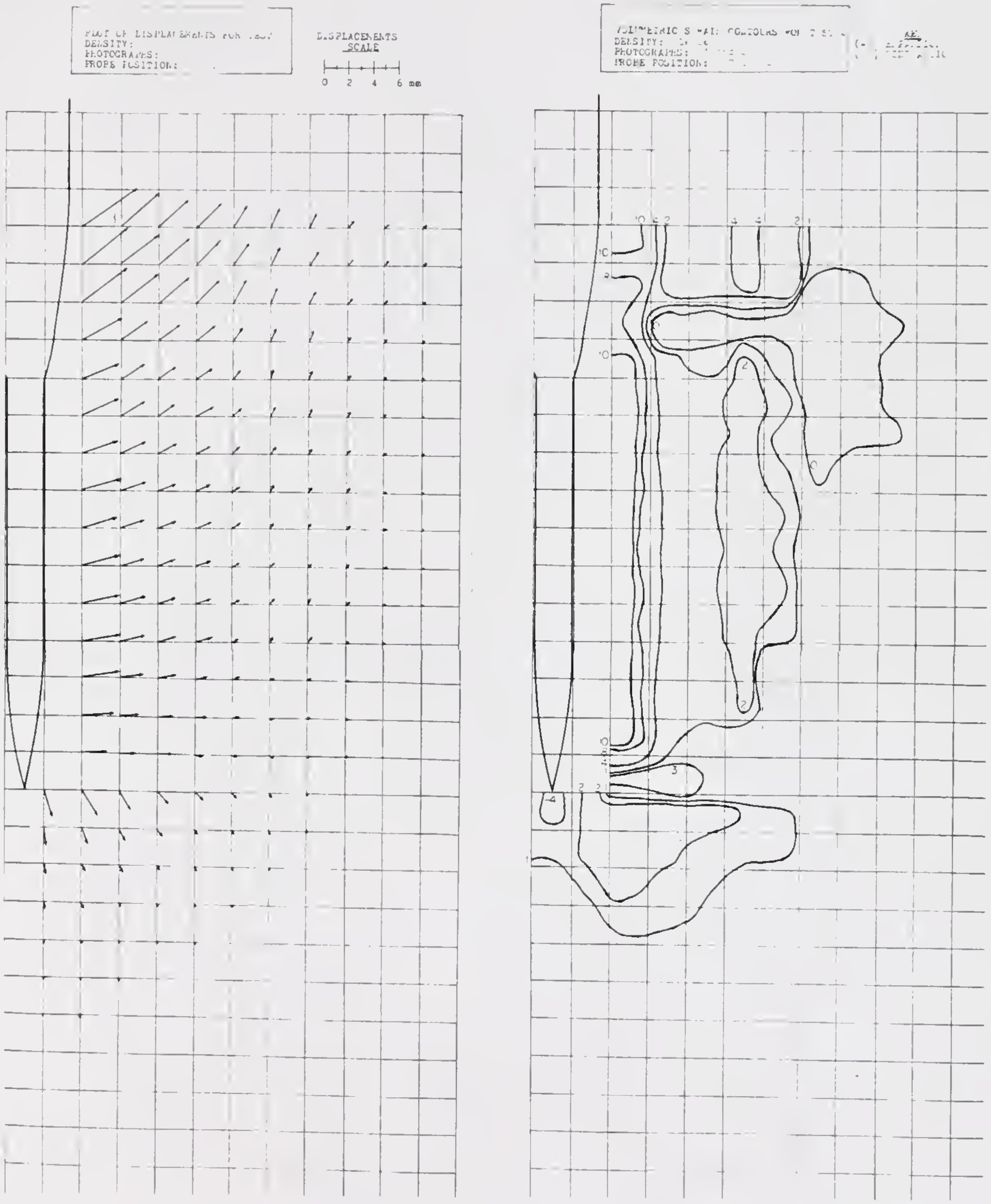


Figure 6-22. Plots of Displacement and Volumetric Strain for Dense Sand

The contours of volumetric strain showed negative (expansion) values at and adjacent to the tip. A negative zone also exists at about 90 millimeters from the probe. The contours in the flat area of the probe showed uniform change of volumetric strain.

6.5.5 Very Dense Sand (Figure 6-23)

The stereo viewing showed all x-parallaxes positive indicating movement to the right, y-parallaxes were positive over most of the probe and negative below the tip. Displacements were measurable to a distance of 170 millimeters from the edge of the probe and to a depth of 110 millimeters below the tip. The magnitude of maximum displacement at the flat area of the probe was five millimeters and decreased with distance from the probe.

The contours of volumetric strain show four negative (expansion) zones, close to the flat area of the probe, adjacent to the probe at the ground surface, below and adjacent to the tip. Some negative values were also observed at about 120 millimeters from the probe. The contours showed uniformity of strain changes in the middle of the flat area.

6.6 Comparison of the Dilatometer and Cone Penetration Test Probes

Since identical soils and compaction techniques were used, the current research allows a comparison to be made between the soil behavior around a cone penetration

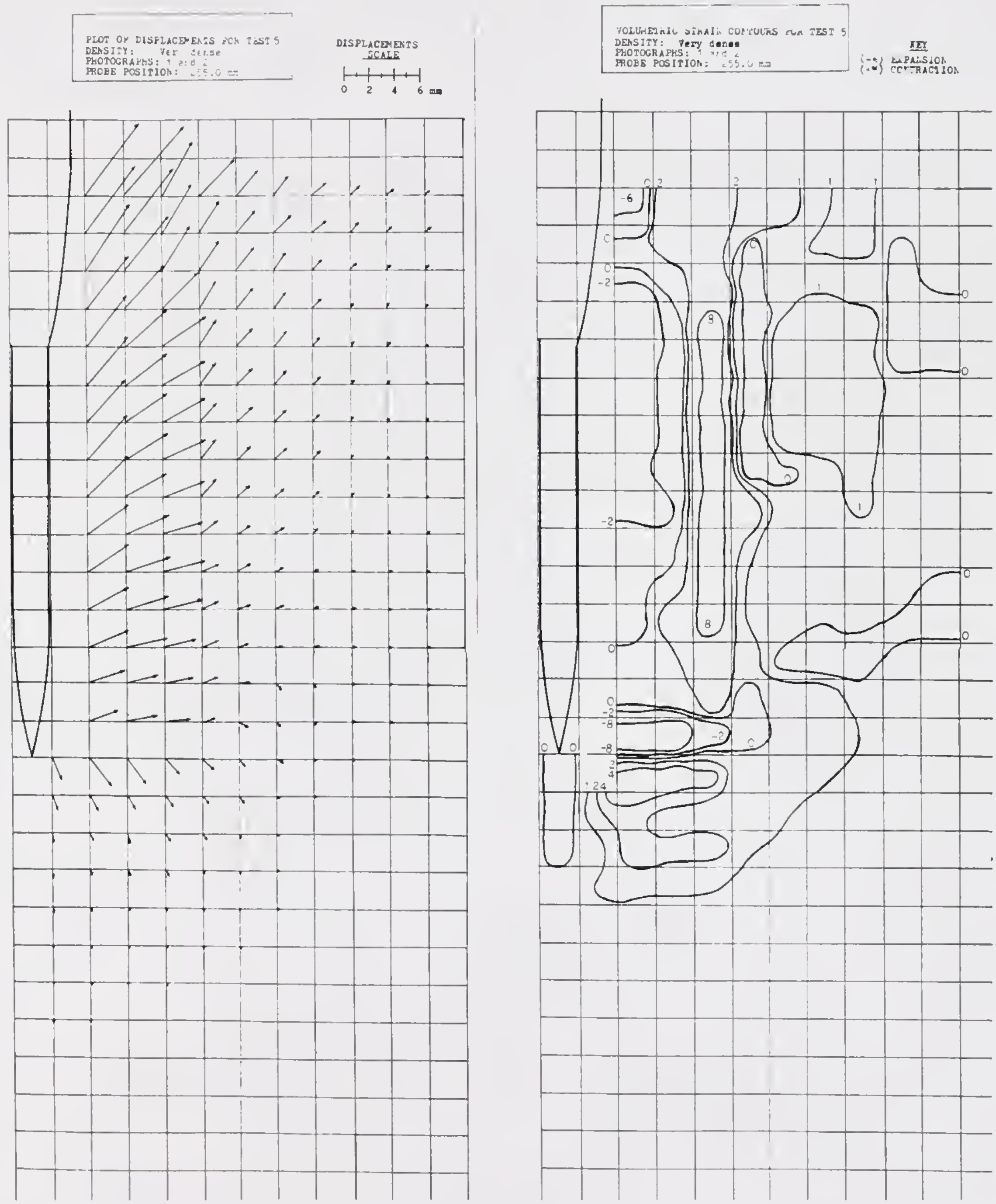


Figure 6-23. Plots of Displacement and Volumetric Strain for Very Dense Sand

test tip and a Marchetti dilatometer or piezoblade. The displacement vector and volumetric strain contour plots for the dilatometer are in Figures 6-19 to 6-23 while the corresponding plots for the cone are in Appendix F, pages 299 to 303.

Displacements near the top of the dilatometer are very similar in magnitude and direction to those of the cone. In the very loose and loose soils, displacements near the surface are down and away from the probe, while in the denser soils they are up and away. This agreement is reasonable since the neck of the dilatometer has a 35 millimeters diameter while the cone is 36 millimeters.

As the thickness of the dilatometer probe decreases the displacements also decrease. In the 14 millimeters thick flat blade section they are somewhat smaller than those for the cone. At the bottom of the probes the cone with its blunter (60°) angle generates larger displacements than the 10° angle dilatometer.

For both probes, all displacements are downward in the two loosest soils, while in the medium, dense and very dense samples, movement is downward near the tips but upward elsewhere. The depth where the direction of displacement changes is different for the two probes. For the dilatometer this point is right at the tip. Only soil below the probe is pushed downward. For the cone, with decreasing density the point of displacement direction change rises up the shaft of the probe. In the very dense sand it is just above the tip, while in the medium sand it is about two cone diameters above the tip. The reason for the differences in dilatometer and cone is that the latter, with its large tip angle and base area, creates a bearing capacity type failure with downward movement of the soil, while the sharper dilatometer penetrates more easily with the soil sliding and shearing along its faces. The lateral and downward extent of the displacements was smaller for the dilatometer, especially over the flat middle section.

The volumetric strain contours for the dilatometer are much more uniform than for the cone. Indeed, the contours show very uniform behavior near the flat face where both dilatometer and piezoblade readings are taken. In contrast, for a piezocone with measuring device at or near the tip, the volumetric strain contours show great irregularities and sharp changes in sign and magnitude.

The overall impression from both the displacement vectors and volumetric strain contour plots is that the sharper tipped, thinner dilatometer penetrates the sand with considerably less disturbance than the larger diameter, larger tip angle cone.

CHAPTER 7

ANALYSIS AND DISCUSSION OF RESULTS

7.1 Introduction

Piezoblade tests, dilatometer tests and cone penetration tests were performed at the five sites described in Chapter 4. Disturbed and undisturbed samples were also obtained from the sites and classification, consolidation and triaxial laboratory tests performed. This chapter discusses and analyzes the field and laboratory test results.

7.2 Piezoblade Results

Tests were performed at all sites using the newly developed piezoblade. No difficulties were experienced with the device during the field testing program. An unexplained failure of the pressure transducer occurred in the laboratory, possibly due to dropping the blade. This has not been replaced.

Two types of piezoblade tests were performed, constant penetration and incremental penetration. In the former, the penetration was continuous except for the pauses to attach additional drill rods. A continuous plot of pore water pressure versus depth was output. In the incremental test, penetration was stopped every 20 cm and the excess pore water pressure allowed to dissipate for a short time.

7.2.1 Identification of High Drainage Layers

In highly pervious soils, the excess pore pressures generated during penetration dissipate instantaneously and the output plot obtained during a constant penetration test is simply the hydrostatic pressure. At all test sites the ground water level was measured and the hydrostatic pressure increased linearly with depth ($=\gamma_w Z_w$). There were no complicating subsurface conditions such as artesian layers to affect this distribution.

In many piezoblade soundings the pore pressure plot, which might be higher or lower than hydrostatic, would then plot for a distance on the hydrostatic line, indicating a highly pervious layer of that thickness. To substantiate this, nearby dilatometer soundings were checked. As will be shown in Section 7.3.1 the dilatometer very accurately defines soil type.

Piezoblade test number one (PBT-1), Figure A-1, shows hydrostatic or close to hydrostatic readings from the ground water level at 100 cm to 120 cm and at 380 and 480 cm. Dilatometer test number one (DMT-1), Appendix B page 210, was performed two feet from the piezoblade sounding. It indicates sand to 140 cm and at 200, 220, 380, and 480 cm. Jagged peaks towards the hydrostatic line at 200 and 220 cm possibly indicate thin pervious or semi-pervious seams.

For PBT-2, Figure A-2, the field plot suggests permeable layers between 210 and 230 cm, at 620 and 680 cm and between 840 and 870 cm. DMT-2, Appendix B page 211 located two feet away shows sand to a depth of 260 cm, at 620 and 680 cm and from 800 to the end of the sounding at 860 cm.

At site number three, PBT-4, Figure A-4, shows hydrostatic pressure to 330 cm and at 980 cm. DMT-4, Appendix B page 213, two feet away indicates sand to 320 cm, between 360 and 400 cm and one value at 440 cm which was not picked up by the piezoblade. PBT-5, Figure A-5, shows hydrostatic to 300 cm and at 430 cm. DMT-5, five feet from the piezoblade soundings, gives sand to 380 cm, at 440 cm and from 840 to the end of the test at 880 cm. The piezoblade was not at hydrostatic but was trending towards it below a depth of 850 cm.

7.2.2 Site Uniformity

At sites 1, 3 and 5, where more than one piezoblade sounding was performed, the excess pore pressure versus depth plots were compared to evaluate site uniformity. As discussed in the previous section, the sites are all very variable with depth, showing quite jagged plots. This is also evident from the dilatometer test results in Appendix B. To evaluate lateral variability, the piezoblade tests for each site were plotted together, Figures 7-1, 7-2, and 7-3.

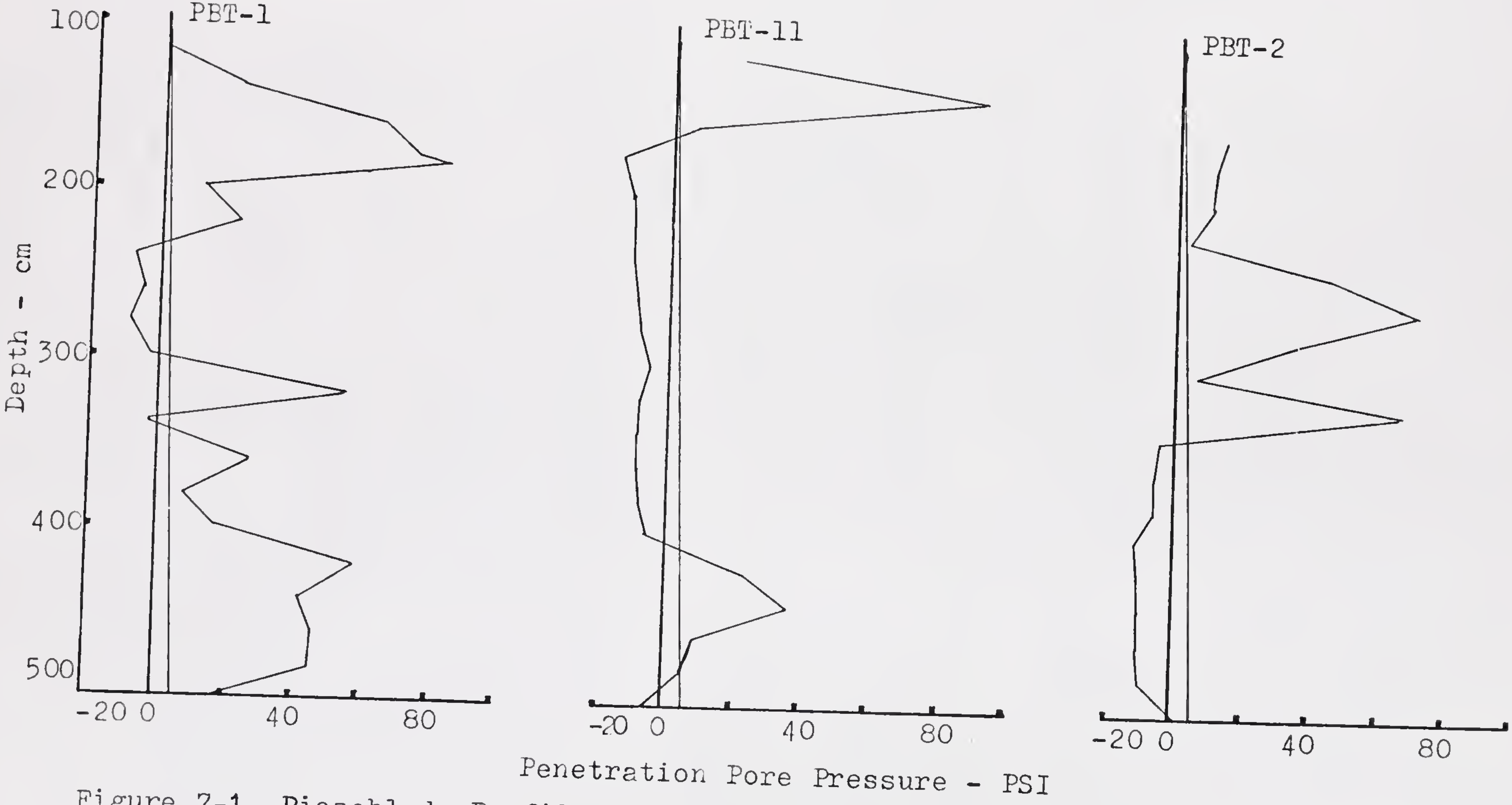


Figure 7-1 Piezoblade Profiles Across Site-1

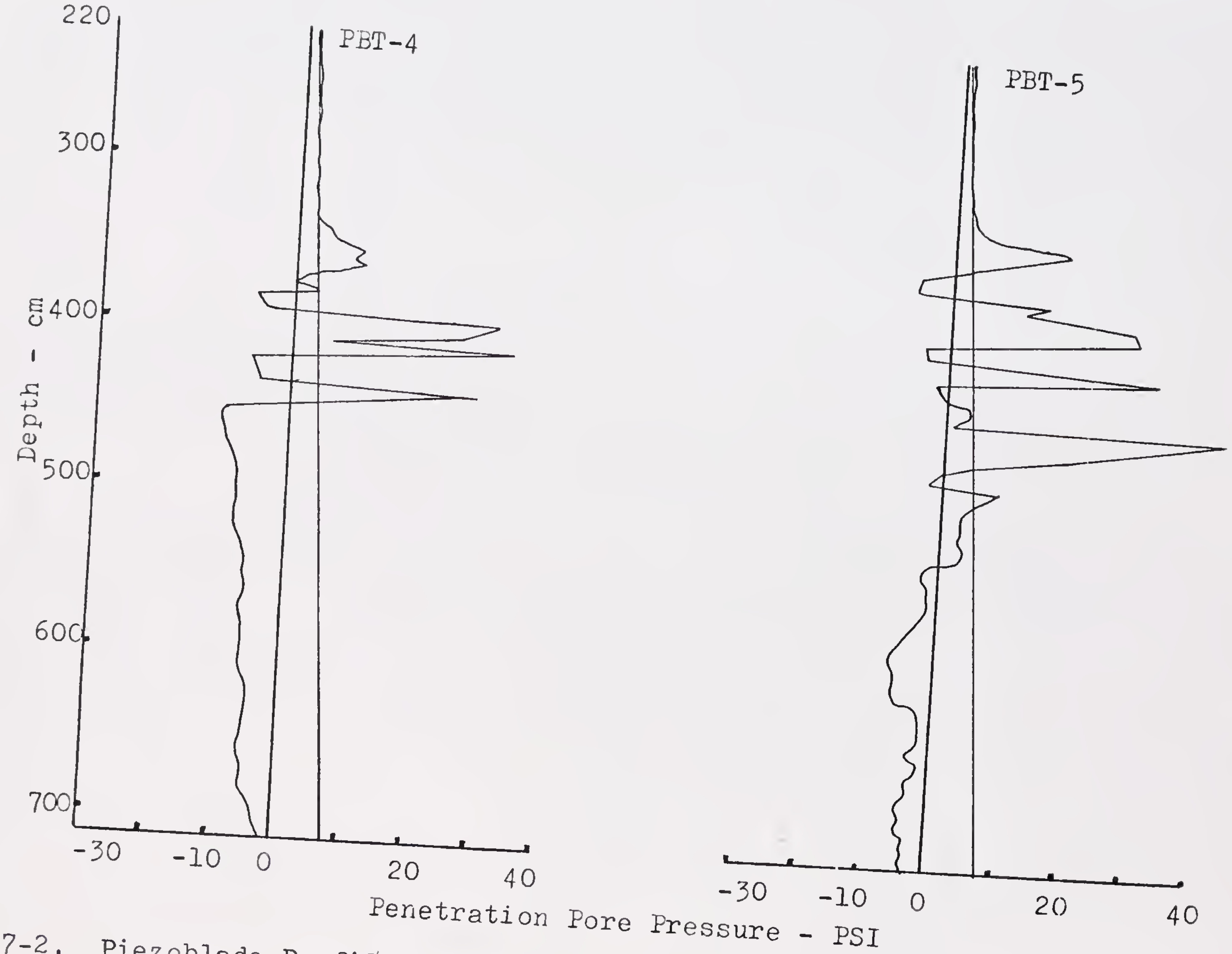


Figure 7-2. Piezoblade Profiles Across Site-3

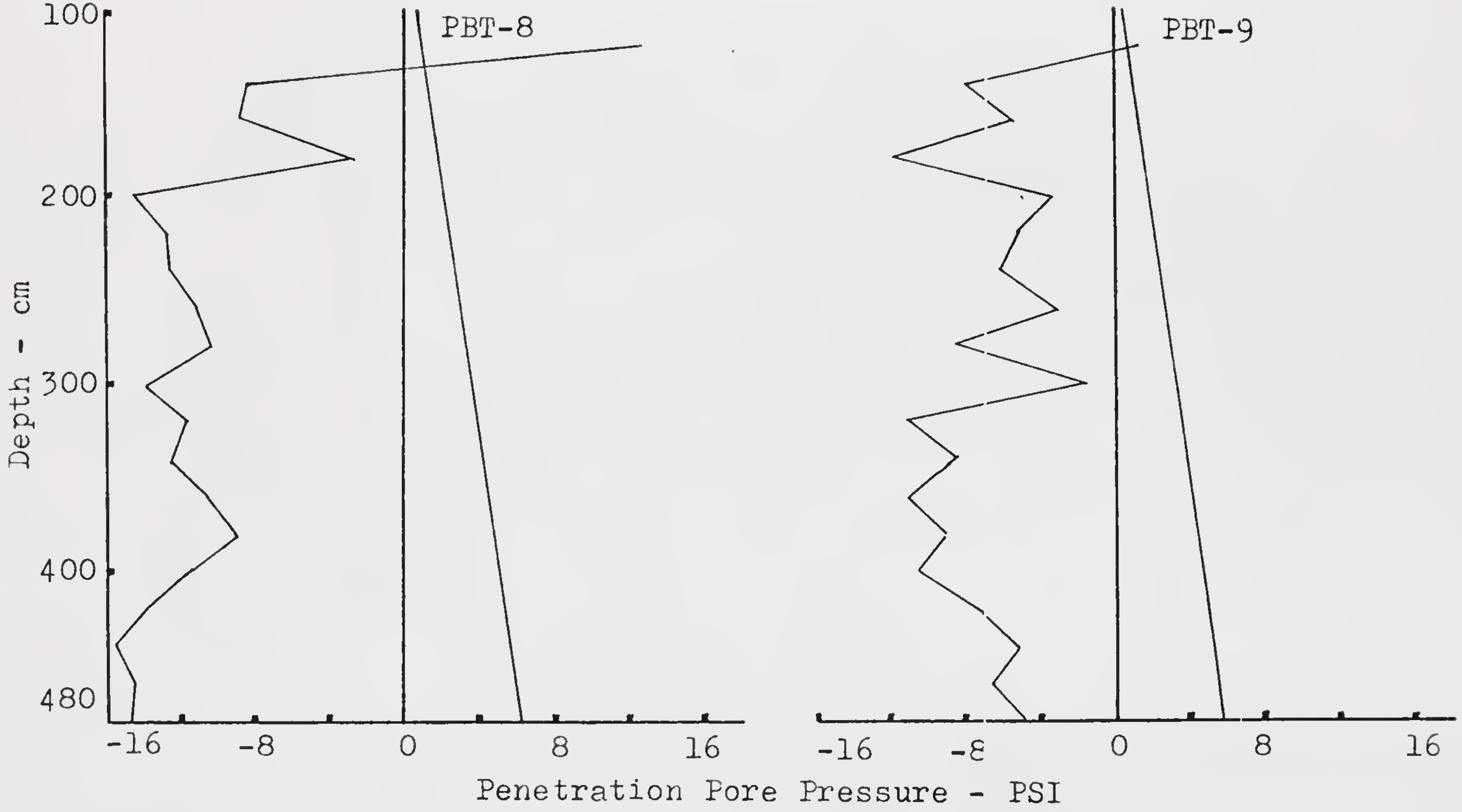


Figure 7-3. Piezoblade Profiles Across Site-5

Only at Site-3, Figure 7-2, where profiles PBT-4 and PBT-5 were located 36 feet apart, was there any similarity between plots. In each of these profiles there are four high positive peaks at approximately the same depth. At Site-5, Figure 7-3, both profiles PBT-8 and PBT-9 were entirely negative below 130 cm but there was no correspondence of peaks. At Site- 1 , Figure 7-1, the three piezoblade profiles were quite dissimilar. As noted in Chapter 4, rapid lateral site variations are to be expected in this geologic region.

7.2.3 Excess Pore Pressure Dissipation Versus Soil Type

In the six incremental piezoblade soundings, penetration was stopped every 20 cm and the excess pore pressures allowed to dissipate for at least one minute. From the closest dilatometer test sounding values of I_D, the material index, were found for the same 20 cm depth increments. Table 7.1 lists all the relevant data; site number, piezoblade sounding number, dilatometer test number, depth, value of I_D from the DMT and percent dissipation of the excess pore water pressure after one minute from the PBT. The one minute was chosen as this is approximately the time required from penetration stop to completion of the dilatometer readings.

Figures 7-4 and 7-5 are respectively a plot of I_D and log (I_D) versus percent dissipation of the excess pore water pressure. Both figures clearly illustrate the

Table 7-1. Percent Dissipation of Excess Pore Pressure in PBT After One Minute

Site No.	PBT No.	DMT No.	Depth (m)	I_D	% Dissipation After One Minute
2	6	3	1.40	1.22	76
			1.60	1.27	23
			1.80	1.10	56
			2.00	1.27	23
			2.20	1.62	16
			2.40	0.95	23
			2.60	0.99	16
			2.80	0.98	--
			3.00	2.15	--
			3.20	1.47	22
4	7	6	1.00	4.77	100
			1.20	8.03	100
			1.40	2.51	94
			1.60	2.88	72
5	8	7	0.80	7.67	100
			1.00	1.43	100
			1.20	1.52	59

Table 7-1. (Continued)

Site No.	PBT No.	DMT No.	Depth (m)	I_D	% Dissipation After One Minute
5	8	7	1.40	1.43	100
			1.60	1.33	34
			1.80	2.11	85
			2.00	1.12	9
			2.20	--	--
			2.40	1.51	13
			2.60	0.67	16
			2.80	0.76	12
			3.00	1.45	28
			3.20	1.18	15
			3.40	1.34	22
			3.60	0.83	13
			3.80	1.59	21
			4.00	1.36	14
			4.20	0.92	10
			4.40	0.77	4
			4.60	--	--

Table 7-1. (Continued)

Site No.	PBT No.	DMT No.	Depth (m)	I_D	% Dissipation After One Minute
5	8	7	4.80	0.73	7
5	9	8	1.20	8.06	100
			1.40	10.81	100
			1.60	9.63	100
			1.80	1.60	52
			2.00	1.55	23
			2.20	0.61	12
			2.40	0.40	7
			2.60	0.69	10
			2.80	0.58	8
			3.00	0.41	7
			3.20	0.17	4
			3.40	0.44	6
			3.60	0.85	13
			3.80	0.46	4
			4.00	0.57	6
			4.20	0.97	17

Table 7-1. (Continued)

Site No.	PBT No.	DMT No.	Depth (m)	I_D	% Dissipation After One Minute
5	9	8	4.40	--	--
			4.60	1.06	12
			4.80	1.03	11
			5.00	1.63	13
			5.20	0.99	12
			5.40	0.91	13
			5.60	0.90	6
			5.80	1.15	9
1	11	11	1.20	2.83	100
			1.40	3.52	82
			1.60	1.76	38
			1.80	1.08	18
			2.00	1.54	17
			2.20	1.38	17
			2.40	1.03	11
			2.60	--	--
			2.80	1.16	10

Table 7-1. (Continued)

Site No.	PBT No.	DMT No.	Depth (m)	I_D	% Dissipation After One Minute
1	11	11	3.00	0.77	10
			3.20	1.11	12
			3.40	0.96	14
			3.60	1.11	11
			3.80	0.80	11
3	13	5	1.90	10.71	100
			2.10	3.49	100
			2.30	4.02	100
			2.50	4.60	100
			2.70	4.77	100
			2.90	3.97	100
			3.10	3.42	100
			3.30	1.70	100
			3.50	1.85	100

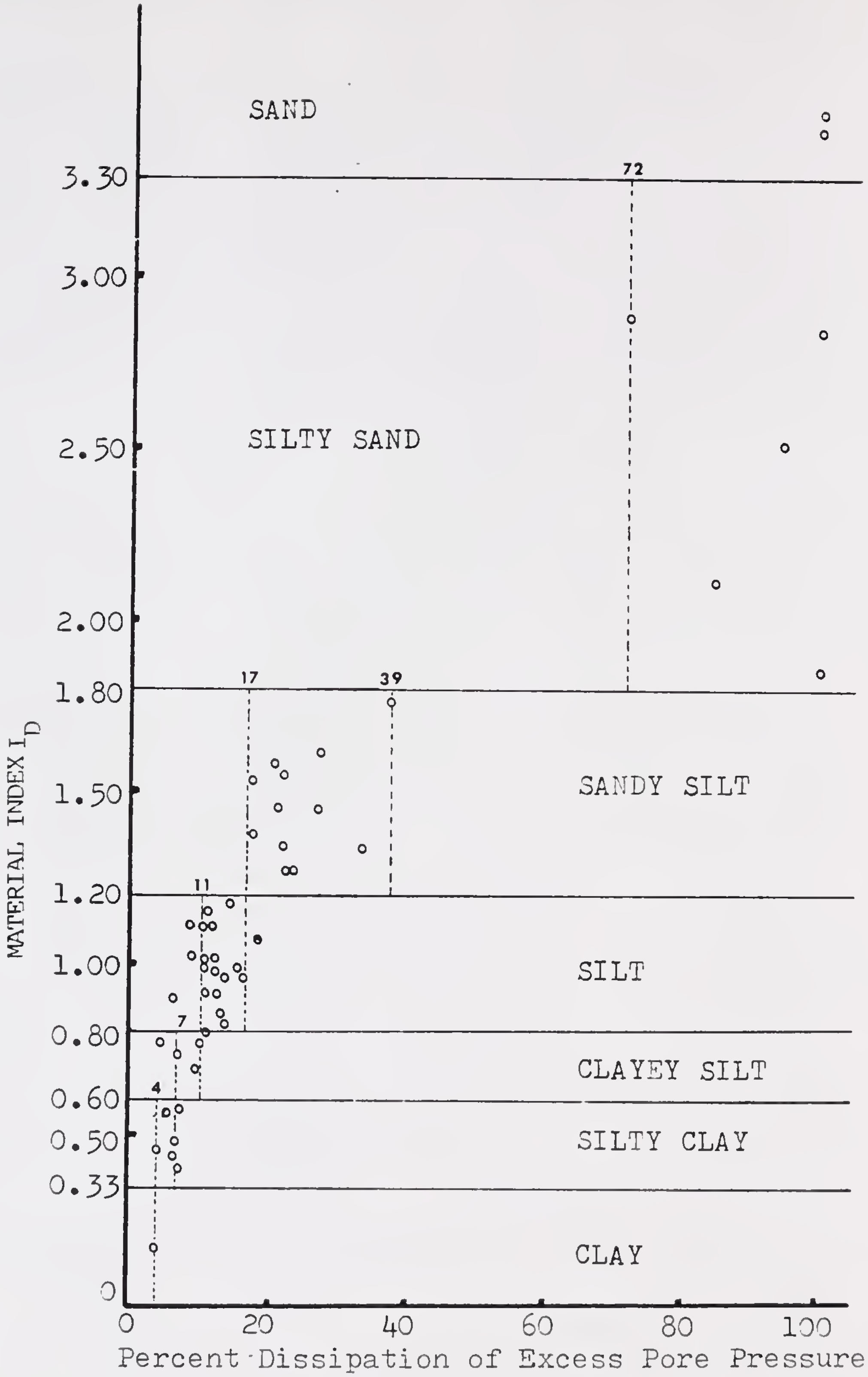


Figure 7-4. Percent Dissipation of Excess Pore Pressure Versus I_D for Different Soils

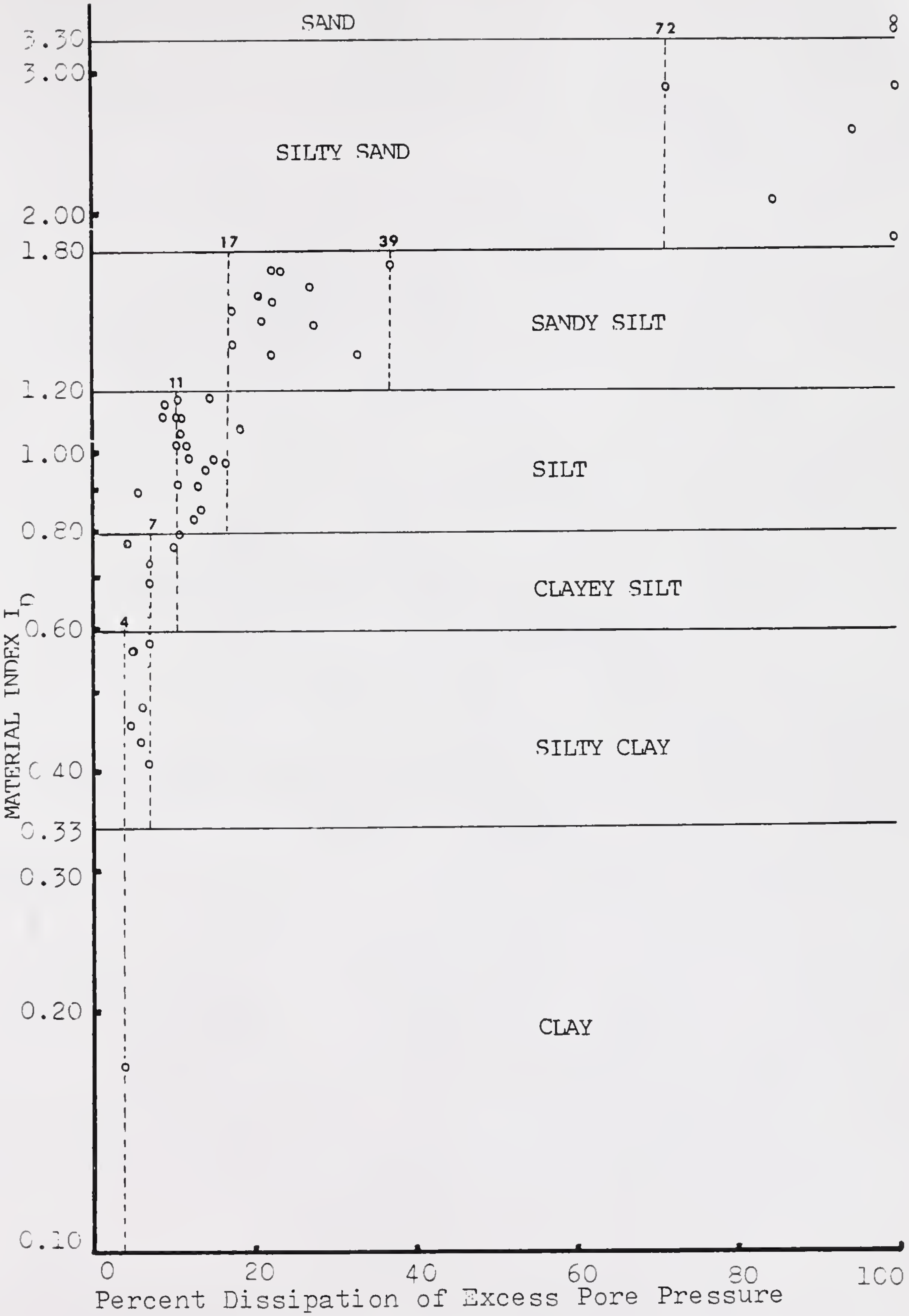


Figure 7-5. Percent Dissipation of Excess Pore Pressure Versus I_D for Different Soils (I_D=log scale)

relationship of increasing percent dissipation with increasing I_D. In Chapter 3, Figure 3-5, the correlation between material index I_D and soil type was given. The seven soil types from this correlation are included in Figures 7-4 and 7-5.

For sand (I_D greater than 3.30) the excess pore pressures dissipated to zero very quickly. For silty sand (I_D between 1.80 and 3.30) the excess pore pressures had dissipated after one minute by between 72 and 100%. The higher the sand fraction, the greater the dissipation. For sandy silts, (I_D between 1.20 and 1.80) the one minute dissipations were between 17 and 39 percent. Some sandy silts however showed a surprisingly high percent dissipation. It was found that these soils were either overlain or underlain (within 20 cm) by a sand layer which would explain the rapid dissipation. Silty soils (I_D between 0.80 and 1.20) showed 11 to 17 percent dissipation, clayey silts (I_D between 0.60 and 0.80) 7 to 11 percent, and silty clays (I_D between 0.33 and 0.60) 4 to 7 percent. Only one clay (I_D between 0.10 and 0.33) soil was tested and it dissipated 4% in the one minute.

The percent dissipation boundaries defined above and determined from limited field testing are included in Figures 7-4 and 7-5 as vertical dashed lines. In actuality there cannot be a gap as shown between 39 and 72 percent. Percent dissipation must be a continuous function. Also the boundaries within each soil type would not be vertical but would have a positive slope, since increased coarser soil fraction results in increased dissipation.

The data show that a dilatometer test giving an I_D value of greater than 3.30, i.e., in sand, will in general be a drained test. For I_D less than 3.30 the test is probably only partially drained, Figures 7-4 and 7-5 giving the relationship.

7.2.4 Initial Excess Pore Pressure Versus OCR and I_D

The dilatometer test provides, at 20 cm depth increments, a value of material index, I_D, which is well correlated with soil type, and a value of the overconsolidation ratio, OCR. The equations for calculating OCR were included in Section 3.2.3. This section compares the initial excess pore water pressures from incremental piezoblade soundings with soil type and OCR values from neighboring dilatometer soundings.

Table 7-2 lists the data; piezoblade sounding number, dilatometer test number, I_D and OCR from the DMT and initial excess pore water pressure from the PBT, while Figures 7-6 to 7-10 are plots of initial excess pore pressure versus overconsolidation ratio for the different soil types.

From Figure 7-6 it can be seen that for all sands and silty sands tested (I_D greater than 1.8) the piezoblade penetration produced positive excess pore water pressures

Table 7-2. Penetration Excess Pore Pressures in Different Soils

PBT No.	DMT No.	Depth (m)	I_D	OCR	ΔU (PSI)
1	1	2.00	2.57	54.45	+9.87
1	1	2.20	2.26	91.45	+22.15
1	1	3.20	1.88	28.04	+51.02
1	1	4.80	2.25	93.92	+41.17
2	2	1.60	2.54	>100.00	+14.34
2	2	1.80	2.30	>100.00	+10.43
2	2	2.00	2.85	>100.00	+8.44
2	2	2.20	2.68	>100.00	+1.29
2	2	2.40	2.55	63.09	+43.00
4	4	3.00	3.29	22.02	0
4	4	3.60	2.02	7.58	+8.44
4	4	3.80	1.83	5.36	0
5	5	2.40	2.89	10.43	0
5	5	3.20	3.08	20.32	0
5	5	3.40	3.24	23.33	+6.45
5	5	3.60	3.09	31.45	+6.15
5	5	3.80	3.16	32.09	+25.30
5	5	4.40	2.43	8.08	-2.98
11	11	1.20	2.40	>100.00	+20.60
11	11	1.40	2.52	>100.00	+90.61
11	11	4.80	2.35	39.41	+.05
12	10	1.60	1.89	76.78	+28.15

Table 7-2. (Continued)

PBT No.	DMT No.	Depth (m)	I_D	OCR	ΔU (PSI)
1	1	1.60	1.76	71.05	+64.71
1	1	1.80	1.58	59.51	+73.01
1	1	2.40	1.43	58.46	--
1	1	2.60	1.39	37.49	-8.13
1	1	3.00	1.55	36.40	-4.13
1	1	3.60	1.56	33.90	+22.73
1	1	4.60	1.57	76.59	+41.45
1	1	5.00	1.66	66.71	+11.31
2	2	3.40	1.57	58.34	-9.26
2	2	4.40	1.38	>100.00	-16.12
2	2	4.60	1.33	90.51	-16.69
2	2	4.80	1.21	33.46	-15.54
4	4	4.60	1.49	8.50	-14.98
4	4	5.00	1.58	13.00	-13.55
4	4	6.20	1.26	6.96	-11.82
5	5	4.00	1.27	4.87	-7.82
5	5	4.20	1.20	4.05	-5.27
5	5	5.80	1.45	7.92	-13.25
6	3	1.40	1.22	>100.00	-6.49
6	3	1.60	1.27	>100.00	-10.20
8	7	1.40	1.43	29.22	-9.57
8	7	1.60	1.35	18.85	-10.13

Table 7-2. (Continued)

PBT No.	DMT No.	Depth (m)	I_D	OCR	ΔU (PSI)
8	7	1.20	1.52	17.34	+11.58
8	7	2.20	1.32	>100.00	-15.28
8	7	2.40	1.51	29.79	-15.56
8	7	3.40	1.34	66.27	-16.69
8	7	3.80	1.59	>100.00	-13.83
8	7	4.00	1.36	99.00	-16.41
9	8	1.80	1.60	37.46	-13.35
9	8	2.00	1.55	>100.00	-5.06
11	11	1.60	1.76	>100.00	+6.03
1	1	2.80	1.04	41.26	-11.27
2	2	3.60	0.86	35.63	-10.41
2	2	4.00	0.99	39.39	-15.27
4	4	5.20	1.00	5.75	-13.26
4	4	5.40	1.07	7.20	-11.83
4	4	5.60	0.98	8.21	-11.83
4	4	6.00	1.15	4.83	-12.69
4	4	6.40	0.99	3.49	-12.11
4	4	6.80	1.11	4.96	-12.67
4	4	7.00	1.16	3.49	-10.39
5	5	5.00	0.86	4.35	-2.41
5	5	5.20	0.82	6.60	-2.40
5	5	5.40	0.84	5.83	-7.55

Table 7-2. (Continued)

PBT No.	DMT No.	Depth (m)	I_D	OCR	ΔU (PSI)
5	5	6.00	0.92	4.47	-12.69
5	5	6.20	0.91	5.82	-8.68
5	5	6.40	1.03	4.23	-10.40
5	5	6.80	0.96	4.58	-12.11
5	5	7.00	1.04	3.41	-11.82
8	7	2.00	1.12	39.03	-16.42
8	7	3.20	1.18	63.09	-15.55
8	7	3.60	0.83	>100.00	-14.98
8	7	4.20	0.92	>100.00	-13.83
9	8	3.60	0.85	32.70	-15.05
9	8	4.20	0.97	50.58	-11.90
9	8	4.40	1.17	44.48	-10.19
9	8	4.60	1.06	76.68	-11.90
9	8	4.80	1.03	90.38	-10.75
11	11	2.40	1.03	50.66	-12.82
11	11	2.60	0.97	42.30	-11.67
11	11	2.80	1.16	49.96	-11.10
11	11	3.20	1.11	40.60	-11.95
11	11	3.40	0.96	22.47	-12.24
11	11	3.60	1.11	30.27	-13.38
2	2	3.80	0.66	28.06	-9.26
2	2	5.00	0.78	29.17	-2.97

Table 7-2. (Continued)

PBT No.	DMT No.	Depth (m)	I_D	OCR	ΔU (PSI)
3	3	3.80	0.70	57.63	-9.05
3	3	6.00	0.66	13.30	-9.75
3	3	6.20	0.62	12.95	-13.60
3	3	6.40	0.79	10.75	-13.59
3	3	6.60	0.73	8.54	-13.60
4	4	5.80	0.68	6.20	-12.11
4	4	6.60	0.69	3.96	-12.42
5	5	5.60	0.80	5.15	-8.69
8	7	2.60	0.67	20.78	-14.13
8	7	2.80	0.76	19.74	-13.84
8	7	4.40	0.77	67.33	-19.68
8	7	4.60	0.62	44.11	-20.12
8	7	4.80	0.73	36.61	-21.11
9	8	2.20	0.61	13.29	-7.06
9	8	2.60	0.68	17.34	-5.92
11	11	3.00	0.77	42.83	-11.10
11	11	3.80	0.80	25.88	-12.81
11	11	5.80	0.67	15.65	-11.10
12	10	2.80	0.77	14.47	-10.84
12	10	3.20	0.74	18.62	-10.84
12	10	3.40	0.73	23.38	-10.56
12	10	4.60	0.76	30.00	-13.69

Table 7-2. (Continued)

PBT No.	DMT No.	Depth (m)	I_D	OCR	ΔU (PSI)
3	3	5.40	0.43	18.18	-11.61
3	3	5.60	0.57	14.61	-11.32
5	5	6.60	0.59	4.36	-10.39
9	8	2.40	0.49	19.71	-8.20
9	8	2.80	0.58	16.31	-11.34
9	8	3.00	0.41	16.09	-3.92
9	8	3.20	0.17	20.26	-14.77
9	8	3.40	0.44	25.00	-14.48
9	8	3.80	0.46	34.85	-13.33
9	8	4.00	0.57	33.42	-14.77
12	10	3.00	0.45	16.65	-11.13
12	10	3.80	0.56	25.26	-9.69
12	10	4.20	0.52	28.82	-7.40
12	10	4.40	0.58	29.92	-10.55
12	10	4.80	0.59	35.84	-11.40
12	10	5.00	0.57	18.59	-11.40
12	10	5.20	0.52	23.84	-11.39
12	10	5.40	0.58	17.11	-12.11

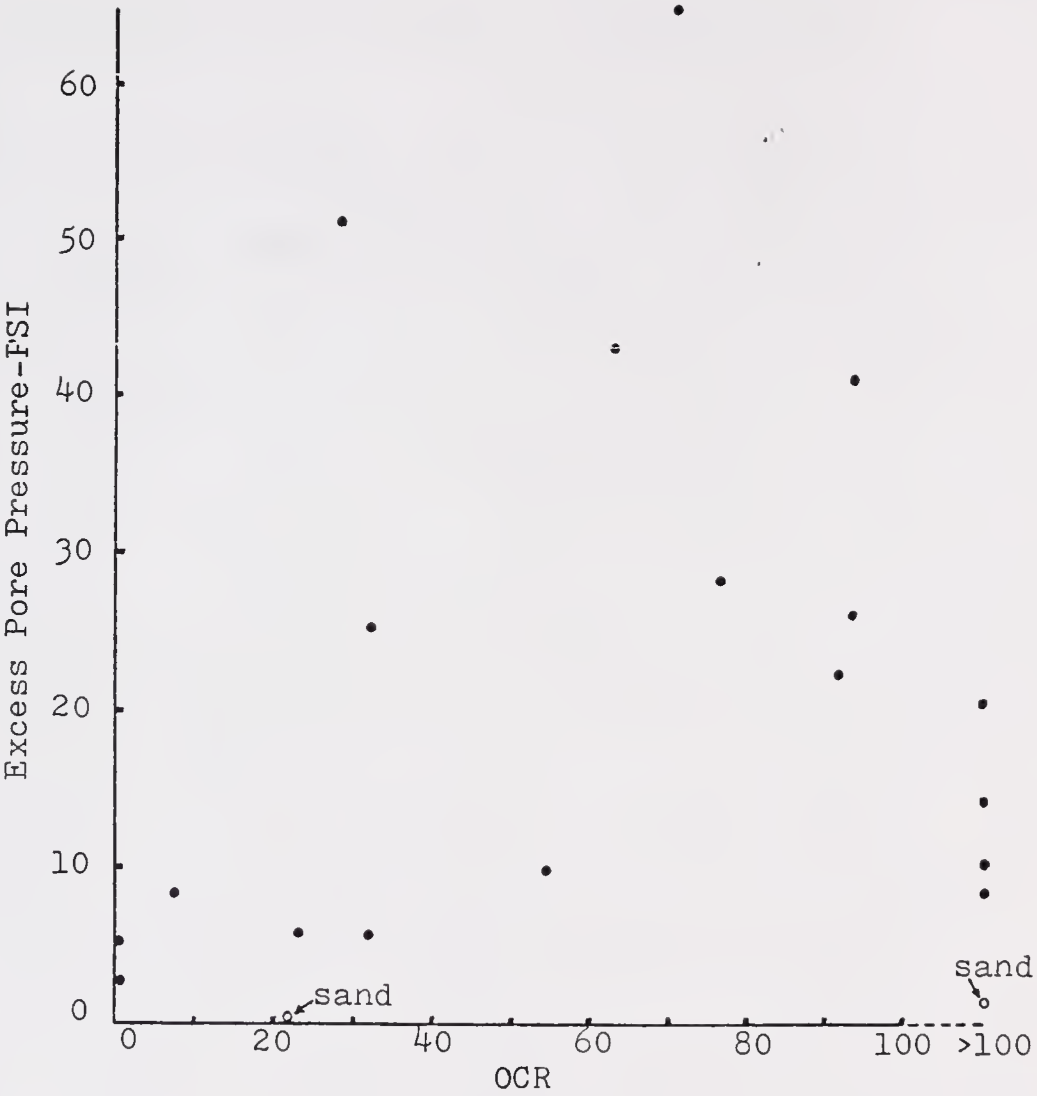


Figure 7-6. Penetration Excess Pore Pressure Versus OCR for Sand and Silty Sand

even for very high overconsolidation ratios. For sandy silts (I_D between 1.2 and 1.8), Figure 7-7, if I_D was less than 1.5 the generated excess pore pressures were negative at least for all OCR's greater than 4 (the only cases available). For I_D greater than 1.6 the excess pore pressures were positive for all values of overconsolidation ratio (as in the sands and silty sands). In the range 1.5 to 1.6 both positive and negative pressures were found. Figure 7-8 shows the data plotted for silt. In all cases, i.e., OCR >3.4, the generated excess pore water pressures were negative. The OCR values plotted on this figure, with the scale 0 to 100, are those calculated from the Marchetti correlation.

$$OCR = (0.5\ K_D)^{1.56} \tag{3.9}$$

As will be described in Section 7.3.2 a better correlation for the Florida soils tested is

$$OCR = (0.372\ K_D)^{1.4}$$

This scale is also included on the figure. The lowest OCR found for any silt was therefore 2 not the 3.4 indicated above.

The initial excess pore water pressure versus overconsolidation ratio plots for clayey silt (I_D between 0.6 and 0.8) and silty clay and clay (I_D less than 0.6) are given as Figures 7-9 and 7-10. In all cases, for the

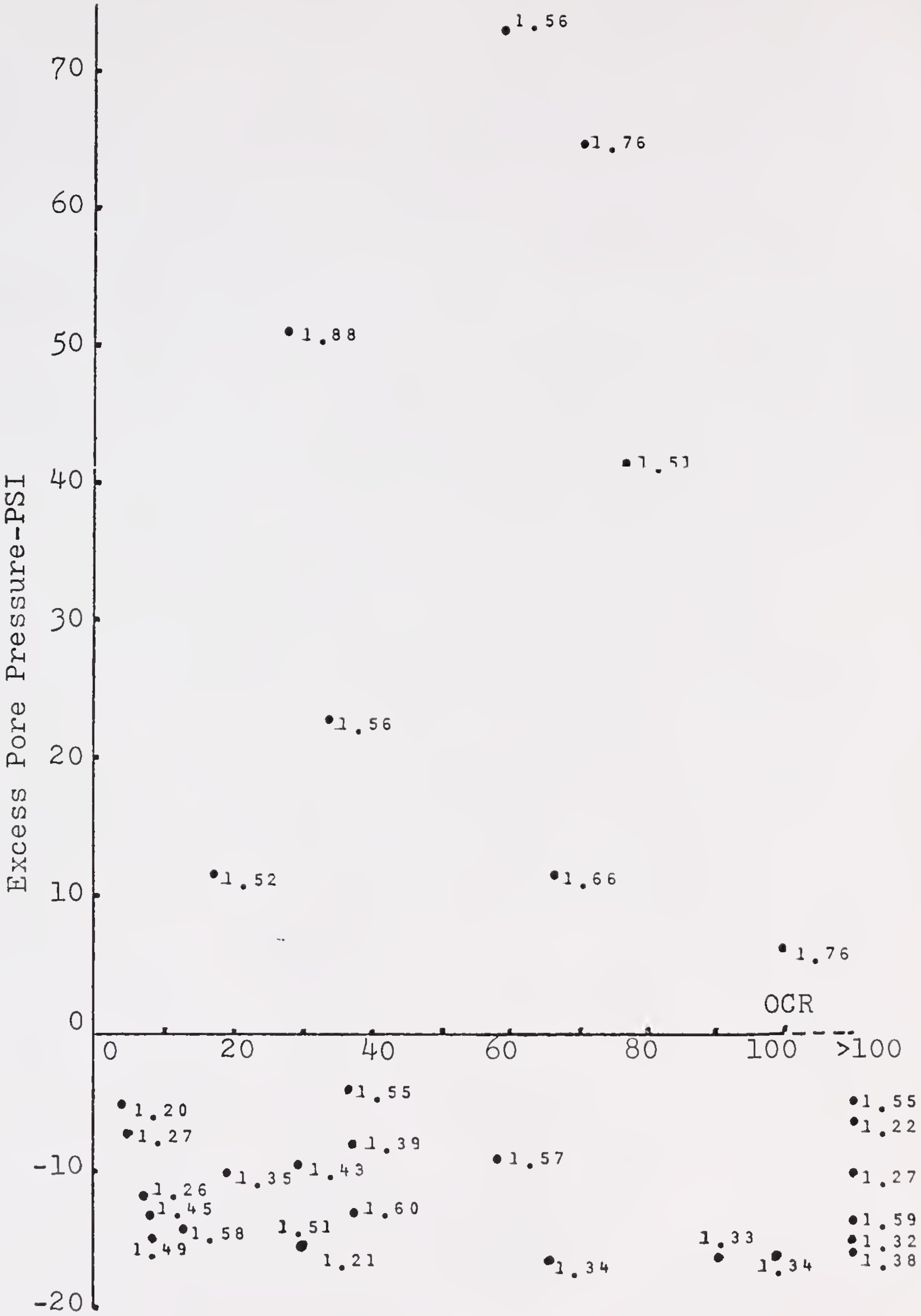


Figure 7-7. Penetration Excess Pore Pressure Versus OCR for Sandy Silt

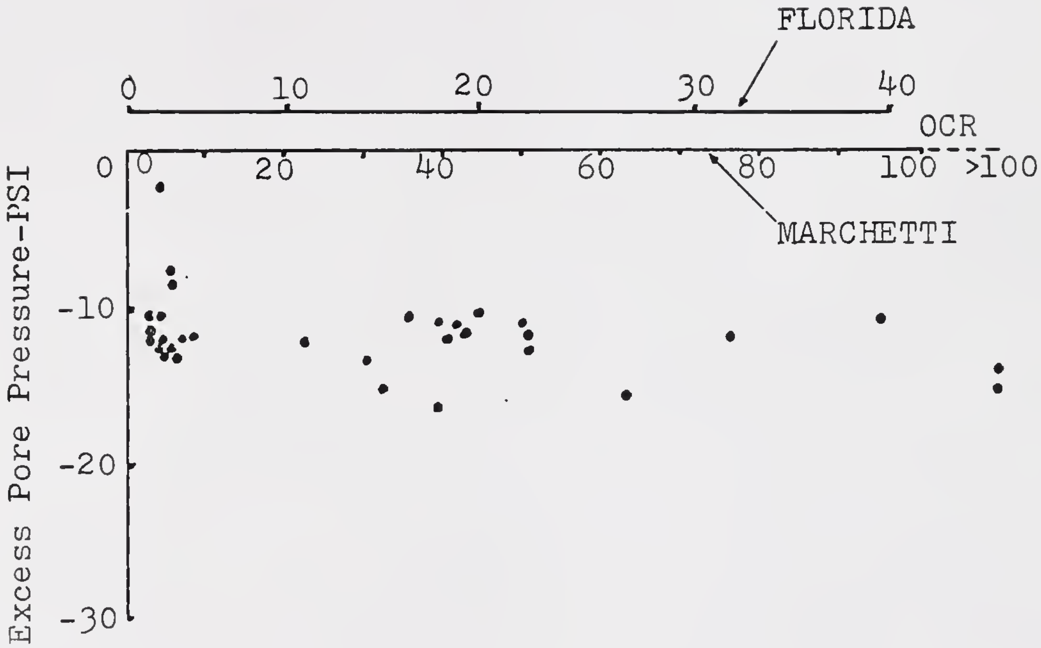


Figure 7-8. Penetration Excess Pore Pressure Versus OCR for Silt

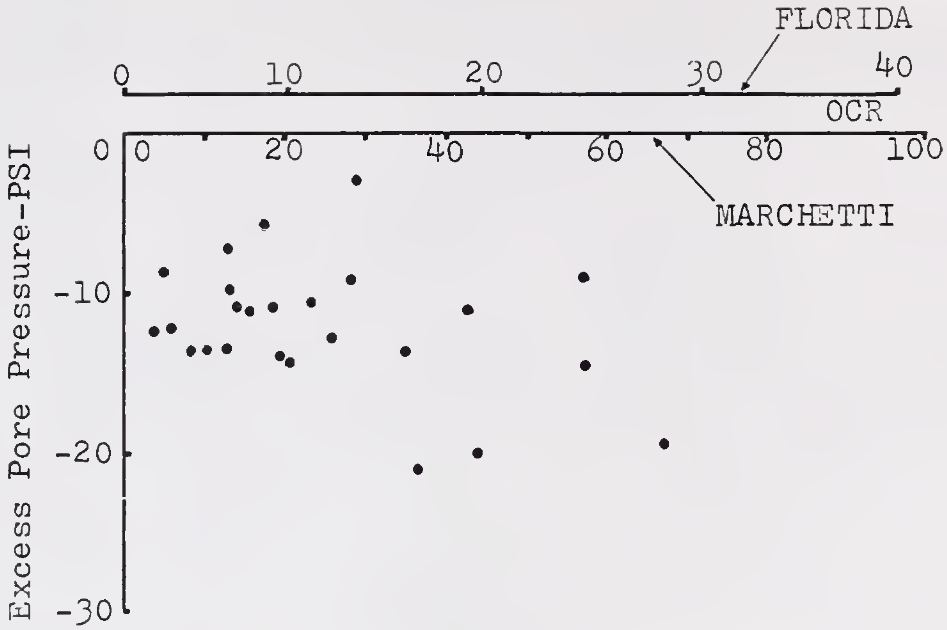


Figure 7-9. Penetration Excess Pore Pressure Versus OCR for Clayey Silt

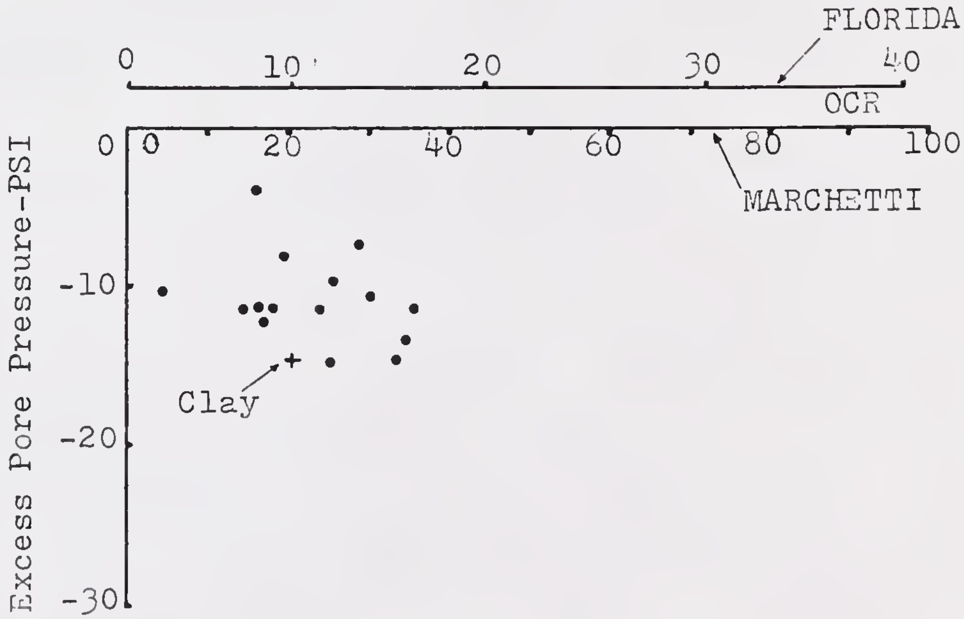


Figure 7-10. Penetration Excess Pore Pressure Versus OCR for Silty Clay and Clay

overconsolidated soils tested the pressures were negative. There is a trend of increasingly negative pressure with increasing overconsolidation. Both the Marchetti and Florida OCR scales are included.

7.2.5 Comparison of Piezoblade and Piezocone

At Site-1 a piezocone sounding was available. This probe, designed by Wissa, was of 36 mm diameter, cylindrical shape with a 60° cone and the small cylindrical porous element attached at the tip. It is described in detail in Davidson (1980b). The test was performed as a continuous penetration test, as in the piezoblade test, stopping only to add rods. The pore water pressure plot is shown in Figure 7-11 along with piezoblade test number one.

It is obvious that there is no systematic agreement between the two tests. The principal reason for this was noted in Section 7.2.2 and Figure 7-1 and is the extreme variability of Site-1. The two tests were approximately 30 feet apart. Piezoblade test number 12 was actually closer than PBT-1 to the piezocone sounding, but was even in greater disagreement, showing primarily negative pressures.

It should be noted that, even if the soil were quite homogeneous, it would not be expected that the plots be identical. The different probe shapes, drainage boundaries and location of the measuring device will all

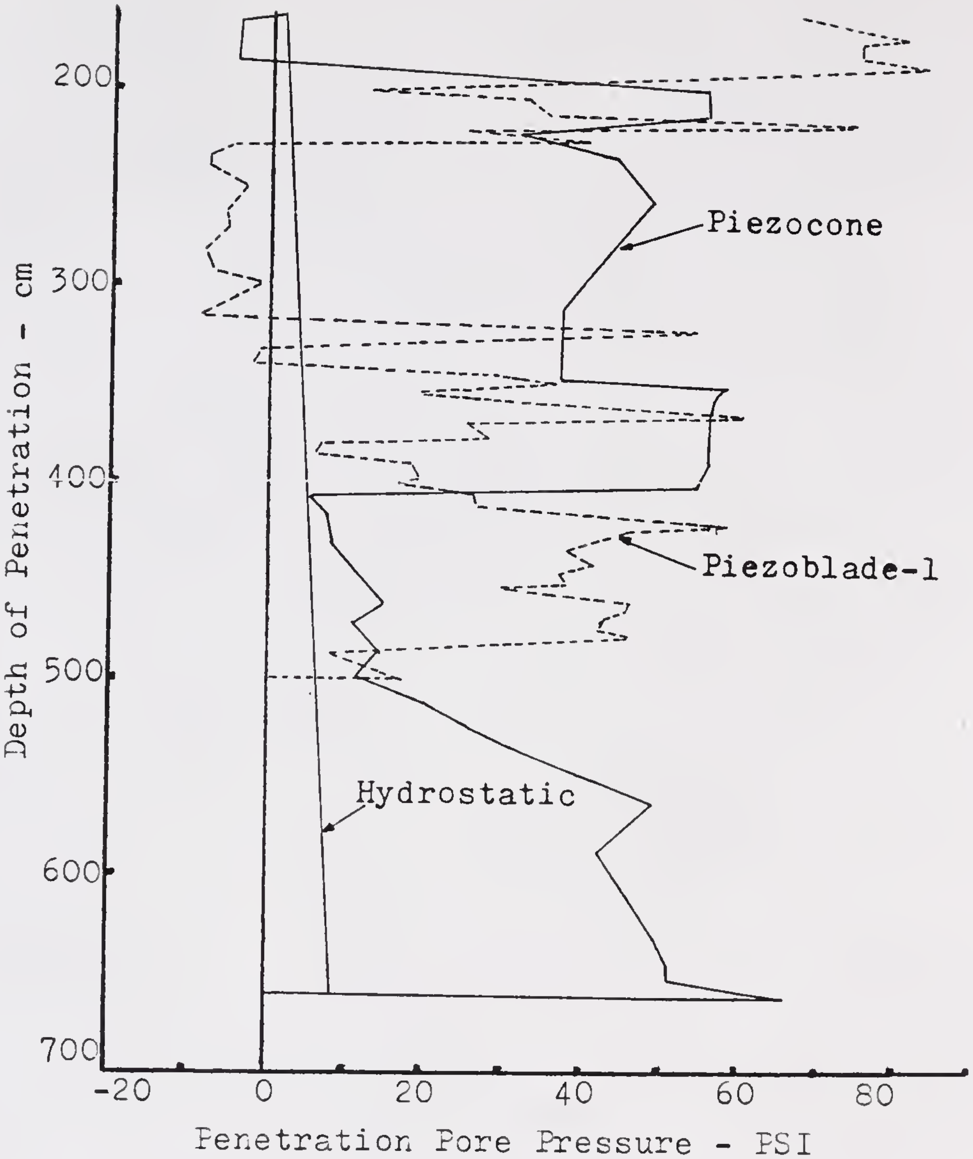


Figure 7-11. Plots of Excess Pore Pressure Versus Depth from Piezocone and Piezoblade Soundings at Site-1

influence the measured excess pore pressures. Some of these effects were observed in the dry sand stereo photography experiment.

7.3 Dilatometer Test Results

Results from the dilatometer tests have already been used in the preceding sections in relation to the piezoblade testing. In this section comparisons will be made between laboratory test results and parameter values obtained by the Marchetti correlations of Section 3.2.

7.3.1 Soil Classification

In geotechnical engineering, soils are usually classified according to the Unified Soil Classification System developed by A. Casagrande (1948). This system, which is described in most soils text books, (e.g., Lambe and Whitman (1969), Holtz and Kovacs (1981)) uses, in general, a two letter classification. The first letter describes the soil type (e.g., S-sand, C-clay) while the second indicates the most important soil property (e.g., W-well graded, H-high liquid limit). To determine a soil's classification a sieve analysis and/or Atterberg limit tests need to be performed. Since soil structure does not play a role in the classification, disturbed but complete samples may be used.

Since soil samples are not acquired during dilatometer or cone penetration testing, correlations have been developed to permit identification of the soils being penetrated. These were introduced in Chapter 3. For the dilatometer the correlation of Figure 3-5 relates soil type and material index, I_D. For the cone penetration test, values of cone bearing and friction ratio allow a determination of soil type from Figure 3-7.

Disturbed samples were gathered from Sites 1, 2 and 3 using a bucket hand auger. Samples were placed in airtight jars and returned to the laboratory for classification testing. Table 7-3 gives site numbers, sample numbers, sample depths and the Unified Soil Classifications determined by laboratory testing. In addition, the table lists the soil type as determined by correlation from neighboring dilatometer and cone penetration soundings. Classification tests were also performed on the undisturbed samples obtained using the Swedish Piston Sampler. Table 7-4 presents this data and the corresponding dilatometer and cone correlation soil types.

A comparison of the field test correlation soil types with the laboratory tested samples shows the dilatometer material index to be a very accurate predictor. Of the 42 samples, 34 were correctly predicted, 2 were classified as sandy silt instead of silty sand and 2 were classified as clayey silt instead of a CL and a CH. The final four were classified as clayey silt instead of clayey sand. This is a consequence of the continuous, increasing I_D - increasing grain size, correlation of

Table 7-3. Soil Classifications by Laboratory, Dilatometer and Cone Penetration Testing

Depth (cm)	Site	Sample No.	Unified Classification	DMT Classification	CPT Classification
50	1	1	SP	sand	organic clay
50	1	2	SP	sand	sand + silt
50	3	3	SP	sand	sand + silt
50	3	4	SP	sand	sand
50	2	5	SP-SM	silty sand	sand
100	1	1	SW-SM	sandy silt	sand + silt
100	1	2	SM-SC	silty sand	sand + silt
100	3	3	SP	sand	sand
100	3	4	SP	sand	sand
100	2	5	SM	silty sand	sandy silty clay
150	1	1	SM	silty sand	sand + silt
150	1	2	SM	silty sand	sand + silt
150	3	3	SP	sand	sand
150	3	4	SP	sand	sand
150	2	5	SM	sandy silt	sandy silty clay
200	1	1	SM	silty sand	sandy silty clay
200	1	2	SM	silty sand	sand

Table 7-3. (Continued)

Depth (cm)	Site	Sample No.	Unified Classification	DMT Classification	CPT Classification
200	3	3	SP	silty sand	sand
200	3	4	SP	sand	sand
200	2	5	SC	sandy silt	clayey sand
250	1	2	SM	silty sand	sand + silt
250	3	3	SP	sand	sand + silt
250	3	4	SP	sand	sand
300	1	2	SC	sandy silt	sandy silty clay
300	2	4	SP	sand	sand

Table 7-4. Soil Classifications by Laboratory, Dilatometer and Cone Penetration Testing

Depth (cm)	Site	Boring No.	Unified Classification	DMT Classification	CPT Classification
200	1	1	SM	silt	clay
363	1	2	CH	silty clay	silty clay
378	1	2	CH	silty clay	silty clay
394	1	2	CH	silty clay	silty clay
455	1	3	CH	silty clay	clay
460	1	3	CL	clayey silt	clay
467	1	3	ML	clayey silt	clay
502	1	3	CL	silty clay	silty clay
440	3	4	SM	silty sand	silty clay
465	3	4	MH	silt	clay
480	3	4	MH	silt	clay
575	3	5	SM	sandy silt	silty clay
590	3	5	ML	silt	clay
355	2	6	SC	clayey silt	sandy clay
380	2	6	SC	clayey silt	sandy clay
560	2	7	CH	silty clay	clay
600	2	7	CH	clayey silt	sand + silt

Figure 3-5, which cannot provide for a sand-clay mixture.

The cone penetration test correlations did not predict as well as the dilatometer. There could be two reasons for this. First, the dilatometer may simply be a better, more consistent tool for such testing. It is thinner than the cone and has a sharper tip, resulting in less soil disturbance. The second reason is that the dilatometer test is much less dependent on operator experience than the cone penetration test. In the latter, with the Begemann tip used in this research, the operator, while controlling the penetration, must watch one or two gauges and obtain readings just prior to and just after a sudden pressure jump. Much cxperience is required to develop the technique. The operators during this research were not highly experienced. In contrast the dilatometer is simple to read. Penetration of the blade has ceased, the operator is seated at a table with one large gauge to read and pressures are increasing slowly and uniformly. Readings are recorded when the buzzer ceases and then reactivates.

7.3.2 Overconsolidation Ratio

Dr. Marchetti developed an experimental correlation between overconsolidation ratio (OCR) and the dilatometer Horizontal Stress Index (K_D). The sites, in Italy, used for the correlation consisted of mechanically overconsolidated uncemented material deposits. In such a cohesive soil, where the overconsolidation is due to surface erosion (simple unloading), Dr. Marchetti found K_D decreased with depth, more rapidly near the surface and slowly at greater depth, Figure 7-12. In normally consolidated cohesive deposits K_D was in the range 1.8 to 2.3 and was almost constant with depth.

The correlation was formulated by performing laboratory one-dimensional consolidation tests on undisturbed samples to determine the OCR and relating these to the field values of K_D. Marchetti's correlations are:

$$\text{clay} \qquad OCR = (0.50\ K_D)^{1.56} \tag{3-9}$$

$$\text{sand} \qquad OCR = (0.67\ K_D)^{1.91} \tag{3-10}$$

In the current research undisturbed samples were obtained at Sites 1, 2, 3 and 5 using the Swedish Piston Sampler. One-dimensional consolidation tests were performed as described in Section 5.4.1 and the OCR for each test determined. Test plots of e vs. log $\overline{P}$ are included in Appendix D, pages 247 to 260 . From the closest dilatometer test sounding, values of K_D were determined and the OCR calculated using the above Marchetti correlations. Table 7-5 contains these results. It is evident that the existing correlations overpredict the OCR for the Florida soils. The final column of the table indicates the magnitude of the overprediction, on the average by a factor of 2.2. The current data were then plotted on Marchetti's correlation graph, Figure 7-13. The one cohesionless

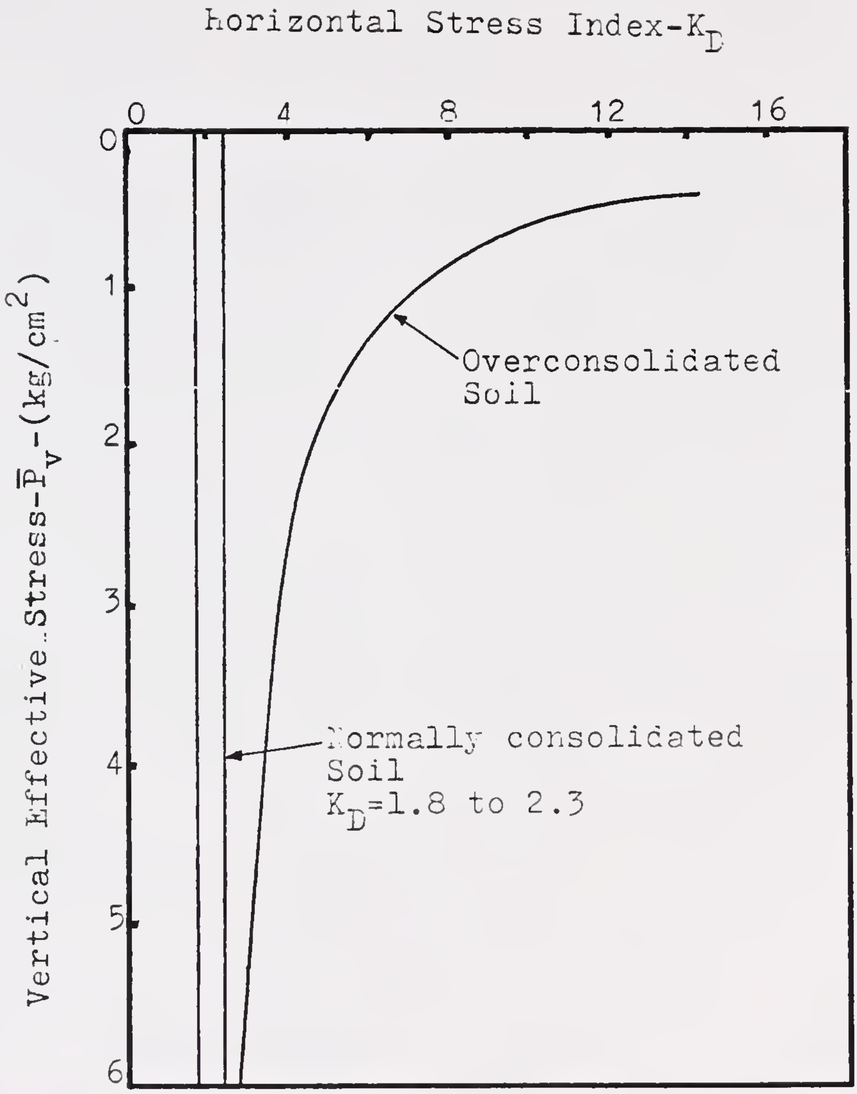


Figure 7-12. Typical Profiles of K_D versus $\overline{P}_v$ for Uncemented Cohesive Soil in Simple Unloading (After Marchetti, 1980)

Table 7-5. OCR's from Oedometer and Dilatometer Testing

Sample No. (Depth) (cm)	Location	Soil Type	OCR from DMT	OCR from Oedometer *	Difference	Over-estimation
US-2 (352)	Site-1	clayey silt	23.38	9.28	14.10	2.52
US-3 (460)	Site-1	clayey silt	30.00	11.50	18.50	2.61
US-3 (495)	Site-1	silty clay	23.00	8.83	14.17	2.60
US-4 (440)	Site-3	silty sand	8.02	7.60	0.42	1.22
US-5 (590)	Site-3	sandy silt	7.92	2.83	5.09	2.80
US-6 (380)	Site-2	clayey silt	57.63	18.15	39.48	3.18
US-7 (560)	Site-2	silty clay	14.61	7.58	7.03	1.93
US-7 (580)	Site-2	silty clay	12.58	7.61	4.97	1.65
US-7 (600)	Site-2	silty clay	13.30	7.45	5.85	1.79

Table 7-5. (Continued)

Sample No. (Depth) (cm)	Location	Soil Type	OCR from DMT	OCR from Oedometer *	Difference	Over-estimation
US-8 (360)	Site-5	silty clay	32.70	18.25	14.45	1.79
US-8 (380)	Site-5	silty clay	34.85	17.51	17.34	2.01
US-8 (400)	Site-5	silty clay	32.42	17.04	16.38	1.96
US-9 (360)	Site-5	silty clay	124.75	47.50	77.25	2.63
US-9 (380)	Site-5	silty clay	82.12	32.15	49.97	2.55
US-9 (400)	Site-5	silty clay	79.88	45.18	34.70	1.77

* Casagrande's method was used to determine the maximum past consolidation stress

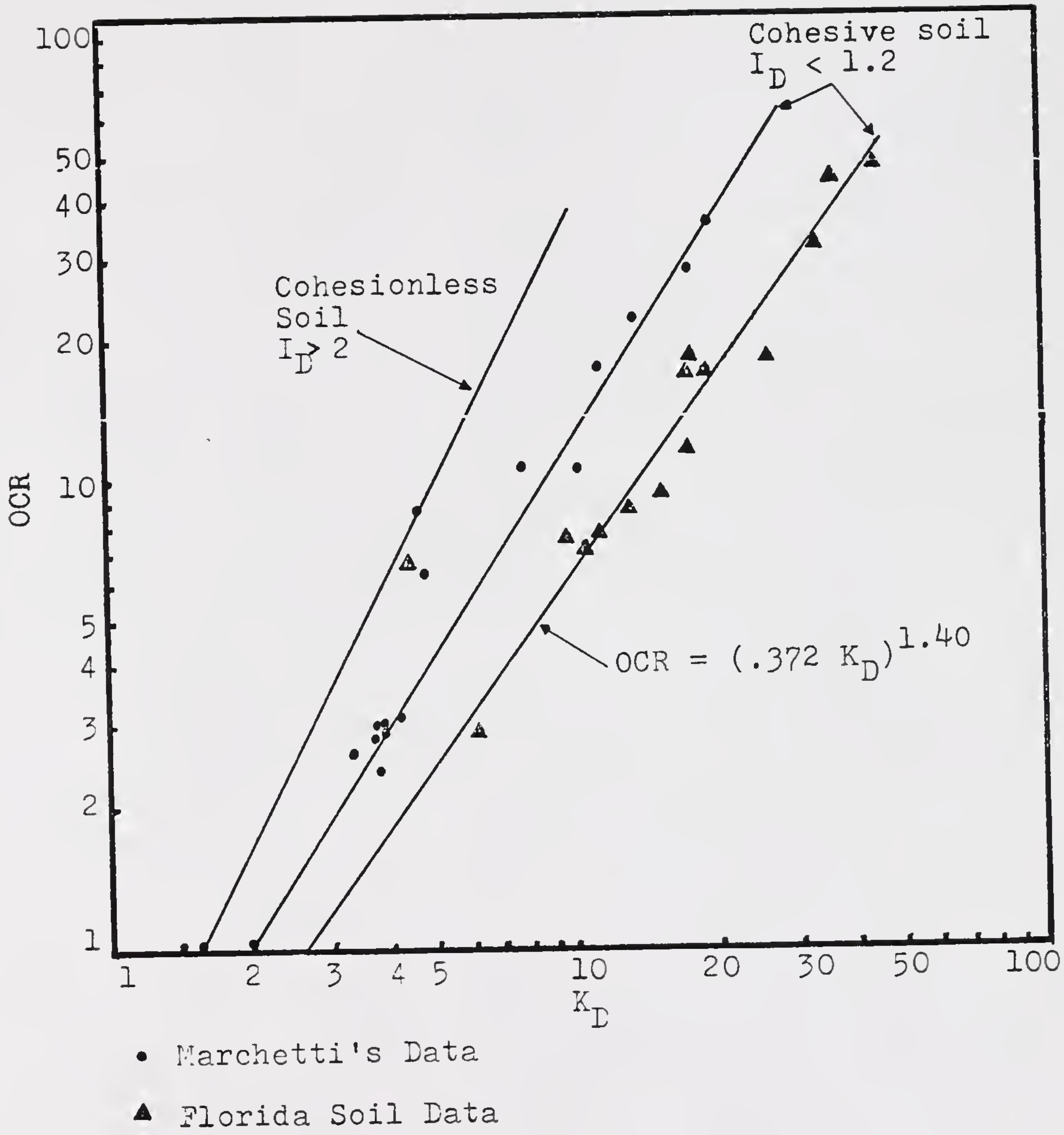


Figure 7-13. Correlations between OCR and K_D - Including Florida Data (After Marchetti, 1980)

sample tested fell close to the Marchetti line. However, the fourteen cohesive samples plotted to the right of the Marchetti data. A best fit straight line through the Florida points gave the correlation equation

$$OCR = (0.372\ K_D)^{1.40}$$

It should be noted that the K_D profiles from the Florida sites were not of the simple form of Figure-12, see data in Appendix B. The overconsolidation is not then the result of a simple unloading.

7.3.3 Constrained Modulus

The constrained modulus (M), defined as change in vertical stress over change in vertical strain during confined compression testing, was also correlated to dilatometer output by Marchetti, (1980). The modulus used was the tangent modulus, i.e., the slope of the stress-strain curve, at the overburden stress. The correlation, listed in Section 3.2.3, related constrained modulus M to two dilatometer parameters, the dilatometer modulus (E_D) and the horizontal stress index (K_D). In the current study, the constrained modulus was found from the laboratory consolidation tests and compared to the value predicted, using the Marchetti correlations and dilatometer parameters from the nearest sounding. Results are included in Table 7-6. Test plots of log M vs. log $\overline{P}_v$ are included in Appendix D, pages 261 to 269. The Marchetti correlation again

Table 7-6. Constrained Moduli (M), from Oedometer and Dilatometer Testing

Sample No. (Depth) (cm)	Location	Soil Type	M - from DMT (bar)	M - from Oedometer (bar)	Difference (bar)	Over-estimation
US-7 (560)	Site-2	silty clay	345.00	177.54	167.46	1.94
US-7 (580)	Site-2	silty clay	440.50	177.21	263.29	2.49
US-7 (600)	Site-2	silty clay	379.90	228.90	151.00	1.66
US-8 (360)	Site-5	silty clay	664.80	335.27	329.53	1.98
US-8 (380)	Site-5	silty clay	423.00	241.94	181.06	1.75
US-8 (400)	Site-5	silty clay	529.70	234.69	295.01	2.26
US-9 (360)	Site-5	silty clay	1335.80	651.37	684.43	2.05
US-9 (380)	Site-5	silty clay	967.90	402.56	565.34	2.40
US-9 (400)	Site-5	silty clay	854.80	373.58	481.22	2.29

overpredicted the laboratory values for these Florida soils. The nine test results were not enough to produce an alternative correlation. All that can be stated is that, on the average, the Marchetti equations overpredicted the constrained modulus for the Florida soils by a factor of 2.1.

7.3.4 Marchetti Modulus

Based on a theory of elasticity solution Dr. Marchetti defined the dilatometer modulus (E_D) as:

$$E_D = \frac{E}{1-\nu^2} = 38.0(P_1 - P_0) \qquad (3\text{-}8)$$

This was derived and the terms defined in Section 3.2.3.

To test the suitability of using dilatometer results to predict Young's Modulus E, a number of consolidated undrained triaxial tests were performed on undisturbed Swedish Piston samples. The laboratory test plots, stress versus strain and pore pressures versus strain, are included in Appendix E, pages 271 to 279 and 280 to 287 respectively. The results are given in Table 7-7; sample identification, the stress and strain values used to calculate the modulus, Young's modulus, dilatometer modulus in psi and bars and the maximum deviator stress and corresponding pore pressure. A Poisson's ratio of 0.50 was used to calculate the dilatometer modulus, since tests were undrained.

Table 7-7. Calculation of Dilatometer Modulus E_D from CU Triaxial Tests

Sample No. (Depth) (cm)	Axial Stress ΔP_a (PSI)	Strain ε	$E=\frac{\Delta P_a}{\varepsilon}$ (PSI)	$E_D=\frac{E}{1-\nu^2}$ (PSI)	E_D (bar)	Maximum Deviator Stress (PSI)	Pore Pressure at Maximum Deviator Stress (PSI)
US-1 (200)	10.00	.0040	2500.00	3333.33	229.90	31.36	-11.14
US-2 (363)	2.50	.00125	2000.00	2666.67	183.90	9.55	-1.25
US-2 (378)	5.00	.0026	1923.00	2564.00	176.82	15.96	-3.10
US-3 (467)	6.00	.0020	3000.00	4000.00	275.86	25.57	-8.80
US-3 (502)	5.00	.0019	2631.58	3508.77	241.98	30.61	-10.60
US-4 (465)	5.00	.0027	1851.85	2469.13	170.29	28.72	-2.90
US-5 (575)	5.35	.0015	3566.67	4755.56	327.97	41.66	-8.00
US-6 (388)	4.99	.0007	7128.57	9504.75	655.50	179.47	-64.20
US-2 (394)	5.00	.0024	2083.33	2777.77	191.60	12.00	--

Table 7-8 shows the comparison between dilatometer moduli determined from the laboratory testing and those calculated from the closest dilatometer soundings using equation (3-8). The final column of the table indicates the factors by which the dilatometer underpredicts the laboratory values for the Florida soils tested. In Figure 7-14 the dilatometer modulus determined by laboratory test is plotted against the corresponding dilatometer sounding $(P_1 - P_0)$ values. A best fit straight line, by the lease square method, was constructed and had the equation:

$$E_D = 52.9(P_1 - P_0)$$

From this, the Young's modulus is calculated by multiplying by $(1 - \nu^2)$.

Table 7-8. Comparison of E_D Values from DMT and CU Triaxial Tests

Sample No. (Depth) (cm)	Location	Soil Type	E_D from DMT (bar)	E_D from CU (bar)	Difference	Underestimation of E_D
US-6 (388)	Site-2	sandy silt	446.80	655.50	208.70	1.47
US-2 (378)	Site-1	silty clay	128.60	176.82	48.22	1.38
US-2 (363)	Site-1	silty clay	128.60	183.90	55.30	1.42
US-3 (502)	Site-1	clayey silt	139.85	241.98	102.13	1.73
US-3 (467)	Site-1	clayey silt	231.50	275.86	44.36	1.19
US-1 (200)	Site-1	silt	169.40	229.90	60.50	1.36
US-5 (575)	Site-3	sandy silt	221.40	327.97	106.50	1.48
US-4 (465)	Site-3	silt	104.90	170.85	65.95	1.62
US-2 (394)	Site-1	silty clay	128.60	190.60	63.00	1.48

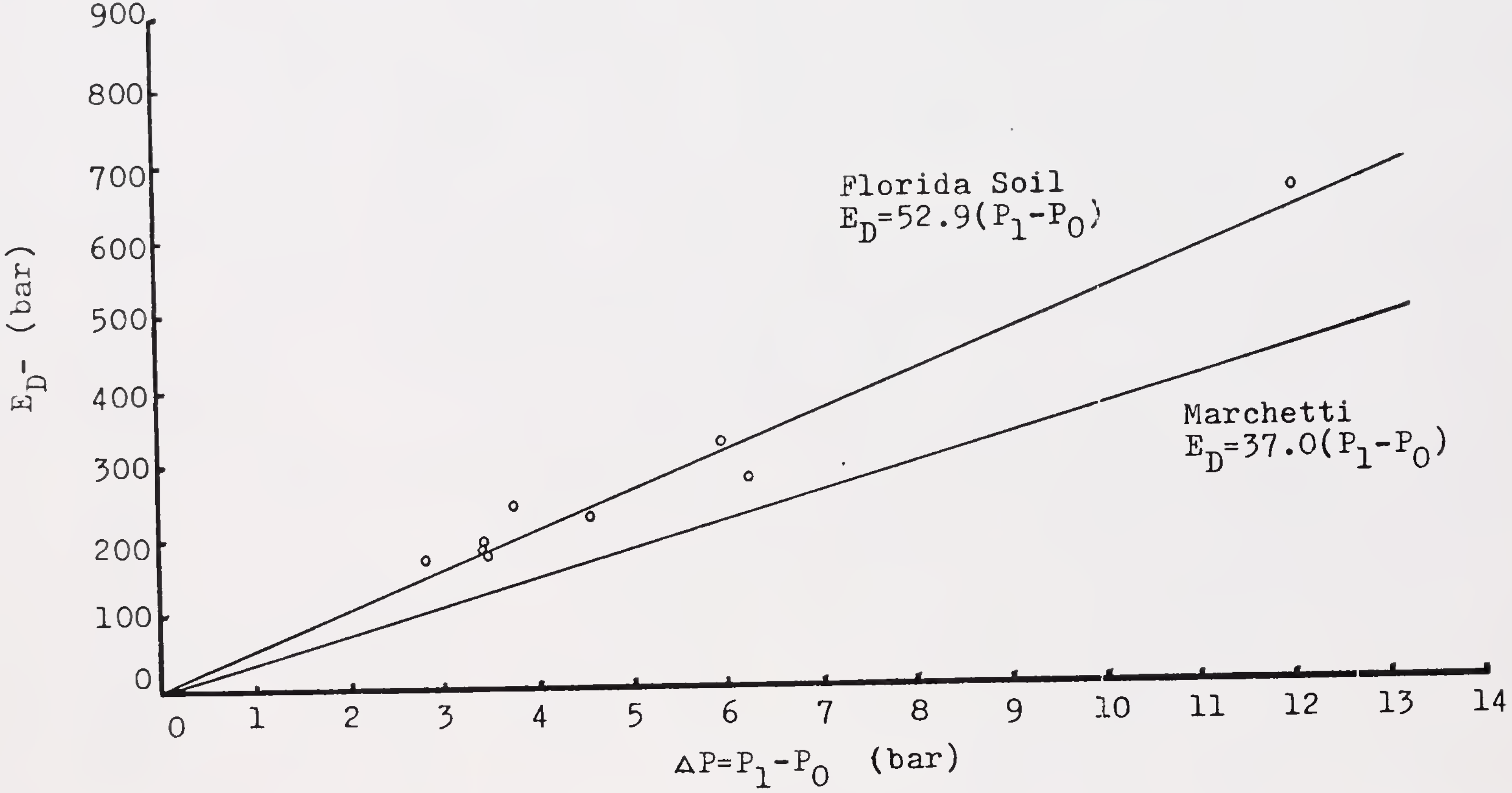


Figure 7-14. Plots of E_D Versus ΔP

CHAPTER 8

SUMMARY, RESULTS AND SUGGESTIONS FOR FUTURE RESEARCH

8.1 Summary

A new soil insitu testing device, called the piezoblade, was designed and constructed. It consisted of a thin steel blade with a flush surface porous cap and an internal pressure transducer. The device is pushed into the ground and pore water pressures, sum of hydrostatic and excess, measured and recorded on a strip chart recorder. A calibration chamber and de-watering system were also designed and constructed.

The new piezoblade serves two purposes. With continuous penetration it is another electric piezoprobe, but of quite different shape from the normal Fugro, Wissa, Geonor, etc. piezocones. The piezoblade was field tested to determine if it provides the same type of data as the piezocone, e.g., detection of permeable layers, positive or negative excess pore pressures in different soils, etc. No attempt was made to compare piezoblade and piezocone results by adjacent field testing.

The second and more important reason for building the piezoblade was to aid in evaluating the drainage conditions around the Marchetti dilatometer. For this reason the piezoblade was built with exterior dimensions identical to the dilatometer. In the field, the tests for comparison with the dilatometer were performed by penetrating the blade discontinuously, in 20 cm increments with a minute or more delay between steps.

The Marchetti dilatometer is a recently introduced insitu testing probe which, by correlations developed in Italy, allows determination of several important geotechnical parameters. There are only a very few dilatometers currently in use in the USA and consequently field experience is limited. A second purpose of this research was to evaluate the available dilatometer correlations in some Florida soils.

Five test sites, with high water table levels, were chosen near the University of Florida campus and an intensive testing program planned. Tests performed included continuous and incremental piezoblade soundings, dilatometer soundings, cone penetration tests and disturbed and undisturbed sampling for laboratory tests. The laboratory testing consisted of classification type tests, specific gravity, grain size and Atterberg limits as well as one-dimensional consolidation and consolidated undrained triaxial tests.

Finally a laboratory study was performed to compare the effects of penetrating blade-shaped and cone-shaped probes. This was carried out in dry sand using a stereo photography technique to accurately measure grain displacement in the vicinity of the probe. Strains could be computed and soil disturbance effects evaluated.

8.2 Results

1. The newly designed and constructed piezoblade, calibration chamber and de-watering system performed satisfactorily.

2. During field testing, the piezoblade successfully and clearly identified thin sand layers. These were confirmed by nearby dilatometer testing.

3. The piezoblade provides a simple and rapid means of evaluating site uniformity, both vertically in a single sounding and laterally, by comparing pore pressure profiles from two or more soundings.

4. By permitting the generated excess pore pressures to dissipate after incremental 20 cm penetrations, an important relationship was established between dilatometer material index I_D (or soil type) and percent dissipation after one minute. This gives a quantitating measurement of the drainage conditions (drained, partially drained, undrained) during dilatometer testing in any soil.

5. The initial excess pore water pressures generated by penetration of the piezoblade were both positive

and negative in sign. Using neighboring dilatometer soundings, relationships were established between excess pore pressure, soil type (I_D) and overconsolidation ratio (OCR). For sands and silty sands the pressures were always positive, even for very high values of OCR. Sandy silts were borderline soils, the high sand fraction gave positive pressures, the low sand fraction gave negative. Silts, clayey silts, silty clays and clays generated negative excess pore pressures. These soils all had an overconsolidation ratio greater than two. There was a general trend of increasingly negative pressure with increasing overconsolidation.

6. A comparison of dilatometer I_D prediction of soil type with the Unified Soil Classification System, determined from laboratory testing on samples, showed excellent agreement.

7. The Marchetti correlation relating overconsolidation ratio and dilatometer horizontal stress index K_D overpredicted the OCR of the soils tested in this study by an average factor of 2.2. A new correlation, better suited to these soils, was developed:

$$OCR = (0.372\ K_D)^{1.40}$$

8. The Marchetti correlation for determining the constrained modulus (M) overpredicted the actual modulus

of the soils tested by an average value of 2.1.

9. The Marchetti equation, based on a theory of elasticity solution, for the modulus E_D underpredicted the values obtained from CU triaxial testing of the Florida soils. The following equation better fits the data:

$$E_D = 52.9(P_1 - P_0)$$

10. In the stereo photography study, the piezoblade was penetrated into five different density sands and displacements and volumetric strains computed and plotted. A similar study had previously been performed on identical soils using a cylindrical-cone probe. A comparison of the results showed that the thinner, sharper tipped dilatometer penetrated the soil with considerably less disturbance than the larger diameter, larger tip angle cone. Such results can be used to compare both dilatometer and cone penetration tests and piezoblade and piezocone tests.

8.3 Recommendations for Future Work

1. One modification of the piezoblade is recommended. In the current model, the transducer holder is permanently affixed and the groove containing the temperature compensating section and wiring filled with a special glue. In the case of a broken wire or the need to recalibrate the transducer it is a tedious task to

remove this glue. Also, the chances of damaging the transducer or wiring is high. It is suggested that a thin plate 40 x 100 mm, flush mounted on a gasket and held by 8 screws, replace the current small circular holder and the glue.

2. The de-aired piezoblade was transported to the site in a water filled plastic bag. This is rather too flimsy and on occasions was accidently punctured by the blade, necessitating a complete re-deairing. It is suggested that stronger plastic containers be produced of approximately the same shape as the blade. (In current piezocone testing at the University of Florida, plastic ketchup and mustard bottles have been successfully used.)

3. Of utmost importance is the locating and testing of better field sites. The sites used in this study, chosen close to the University for reasons of cost and adjacent to lakes for the high ground water levels, were much too variable. Also they consisted primarily of soils ranging from silty sand to silty clay. Homogeneous sites, including deposits of sand and clay are needed.

4. It would be of interest to perform a series of adjacent piezoblade and piezocone tests. All piezometer probes in commercial use are of the piezocone variety.

Different organizations have placed the porous sensing element at different locations near the tip seeking the best position. The stereo photography study in this research however shows great disturbance of the soil in the vicinity of the cone, but very little disturbance along the piezoblade face. Further research may show a complete superiority of the blade shape for piezometric probes.

APPENDIX A

PLOTS OF PIEZOBLADE TEST RESULTS

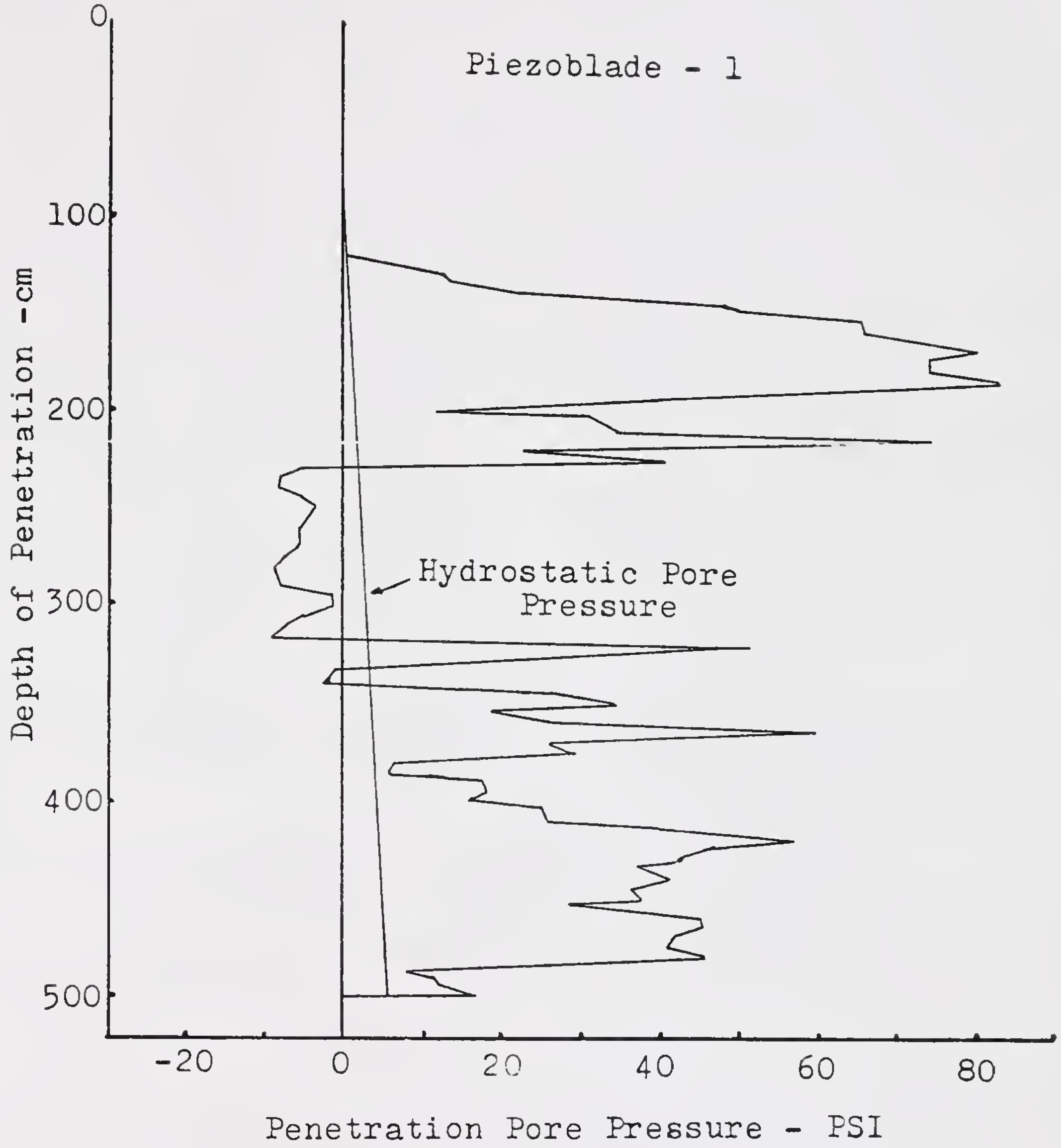
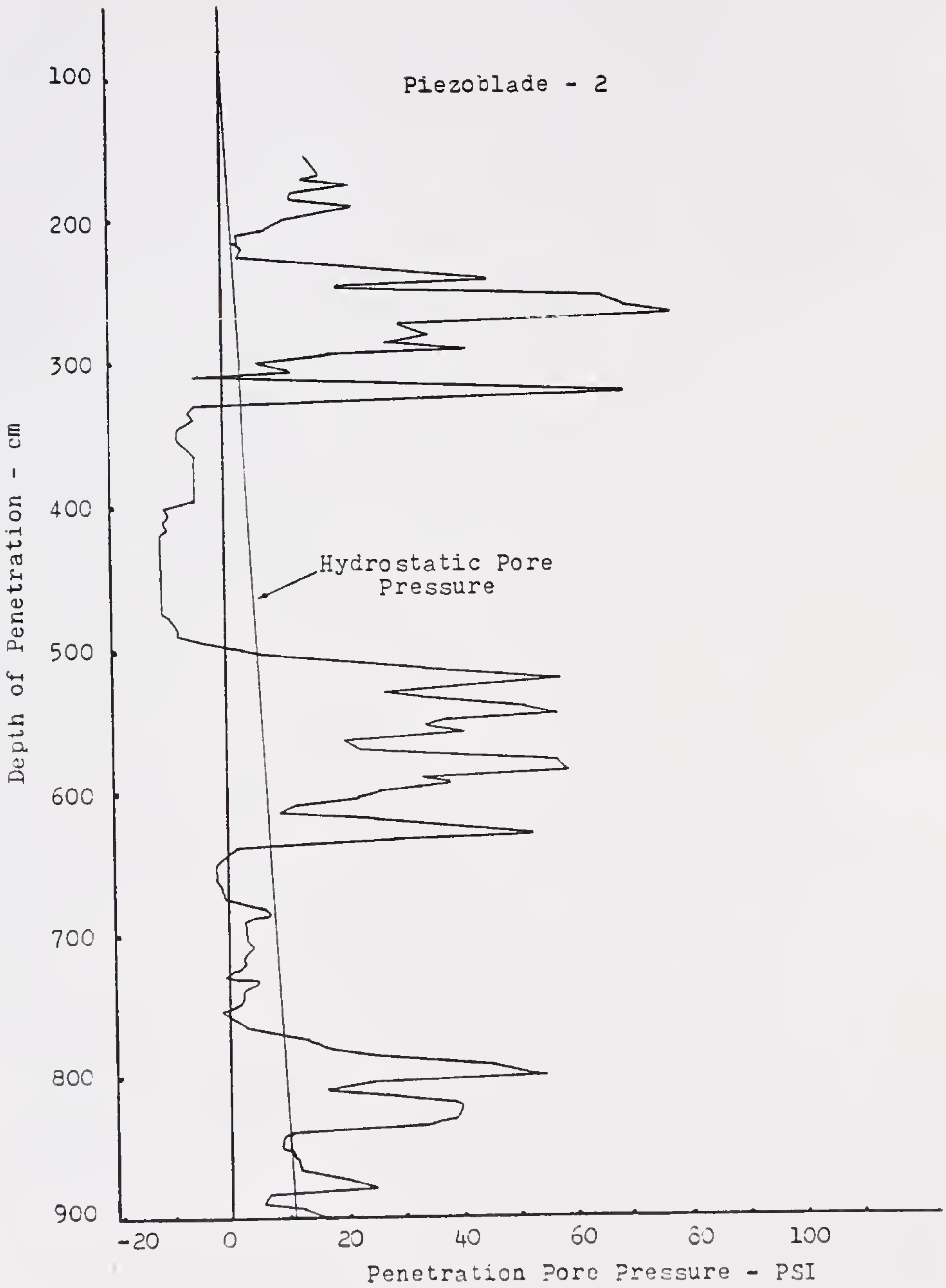


Figure A-1. Pore Pressure Profile from Piezoblade-1 (Continuous Penetration)



Fgiure A-2. Pore Pressure Profile from Piezoblade-2 (Continuous Penetration)

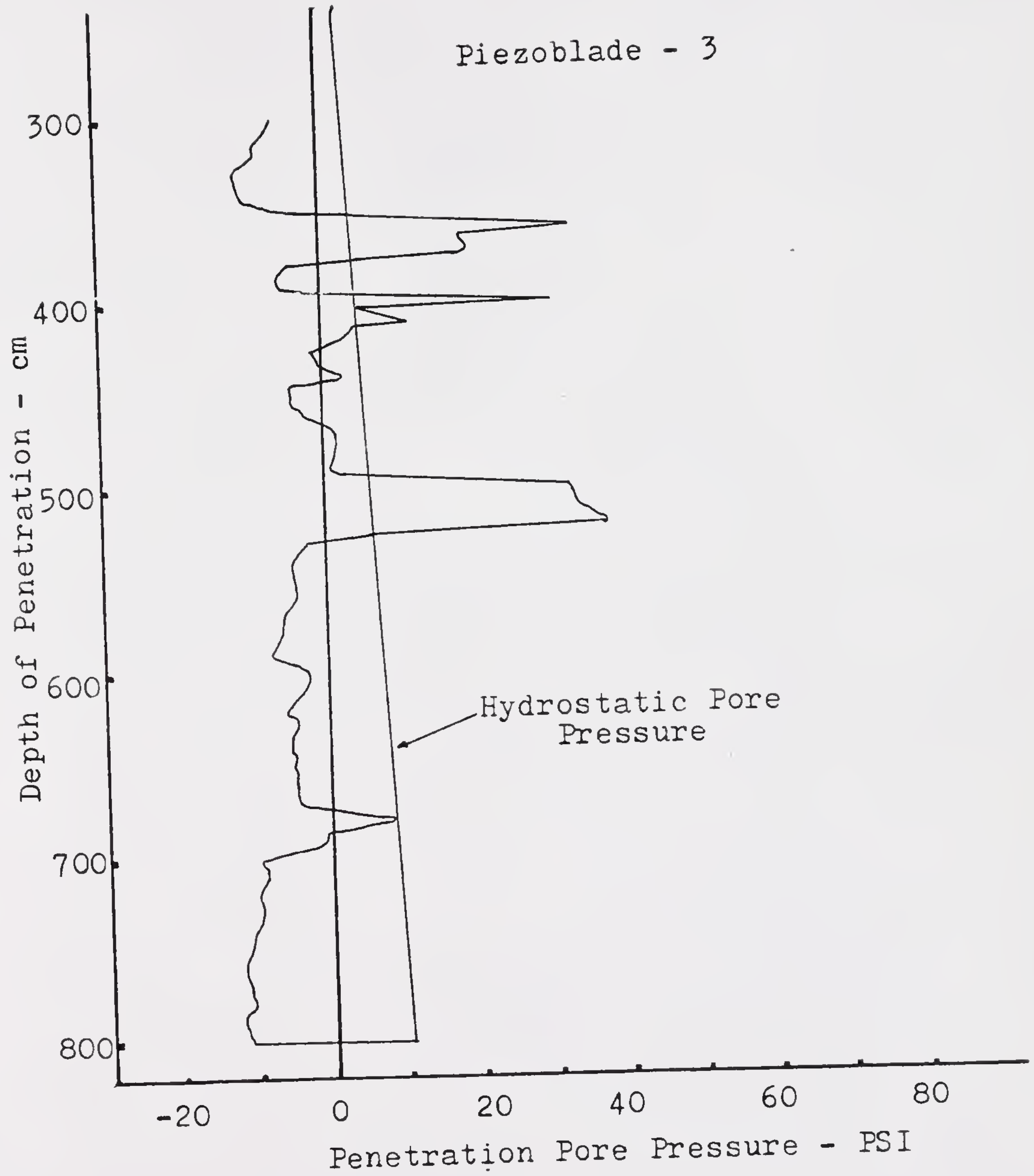


Figure A-3. Pore Pressure Profile from Piezoblade-3 (Continuous Penetration)

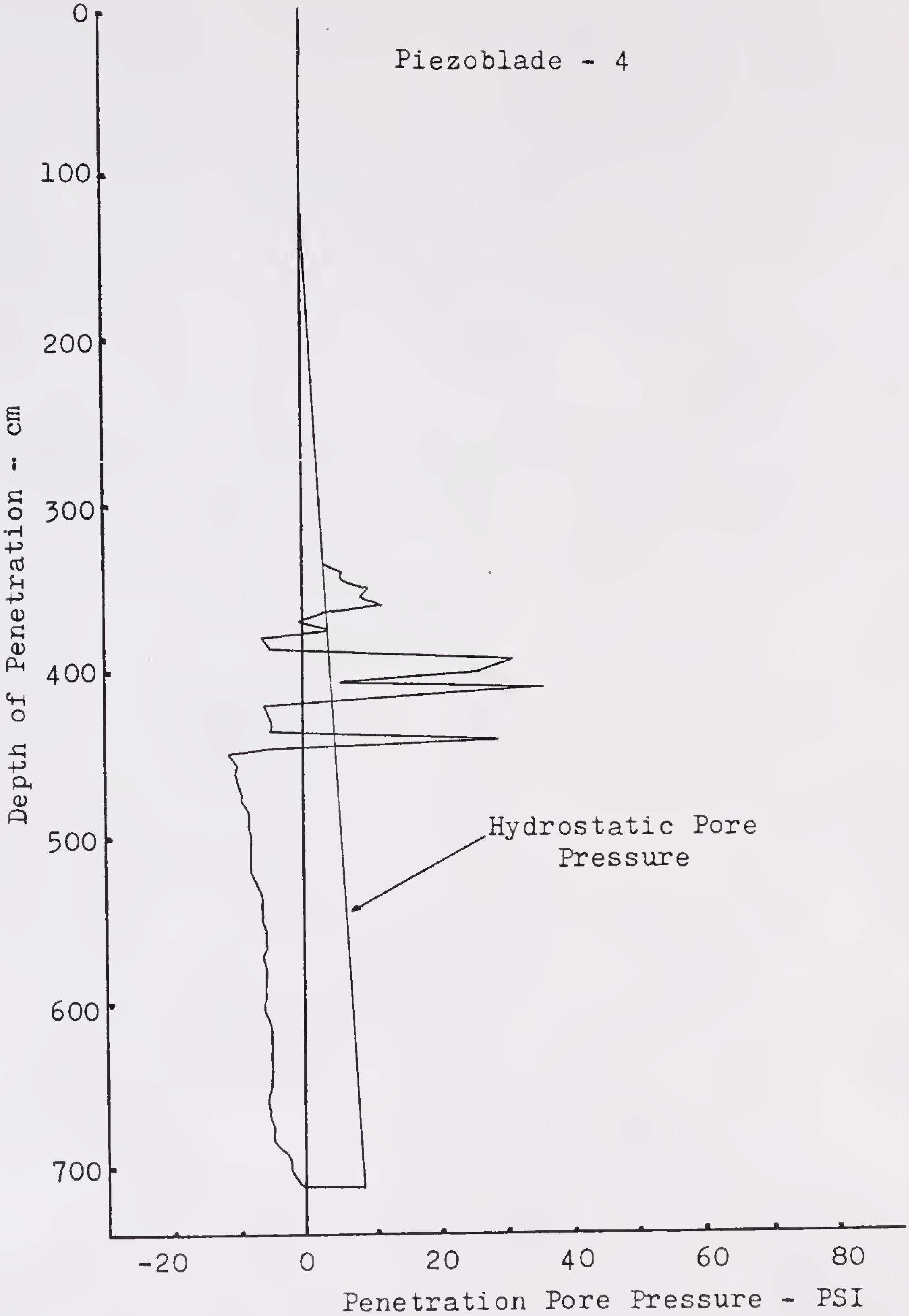


Figure A-4. Pore Pressure Profile from Piezoblade-4 (Continuous Penetration)

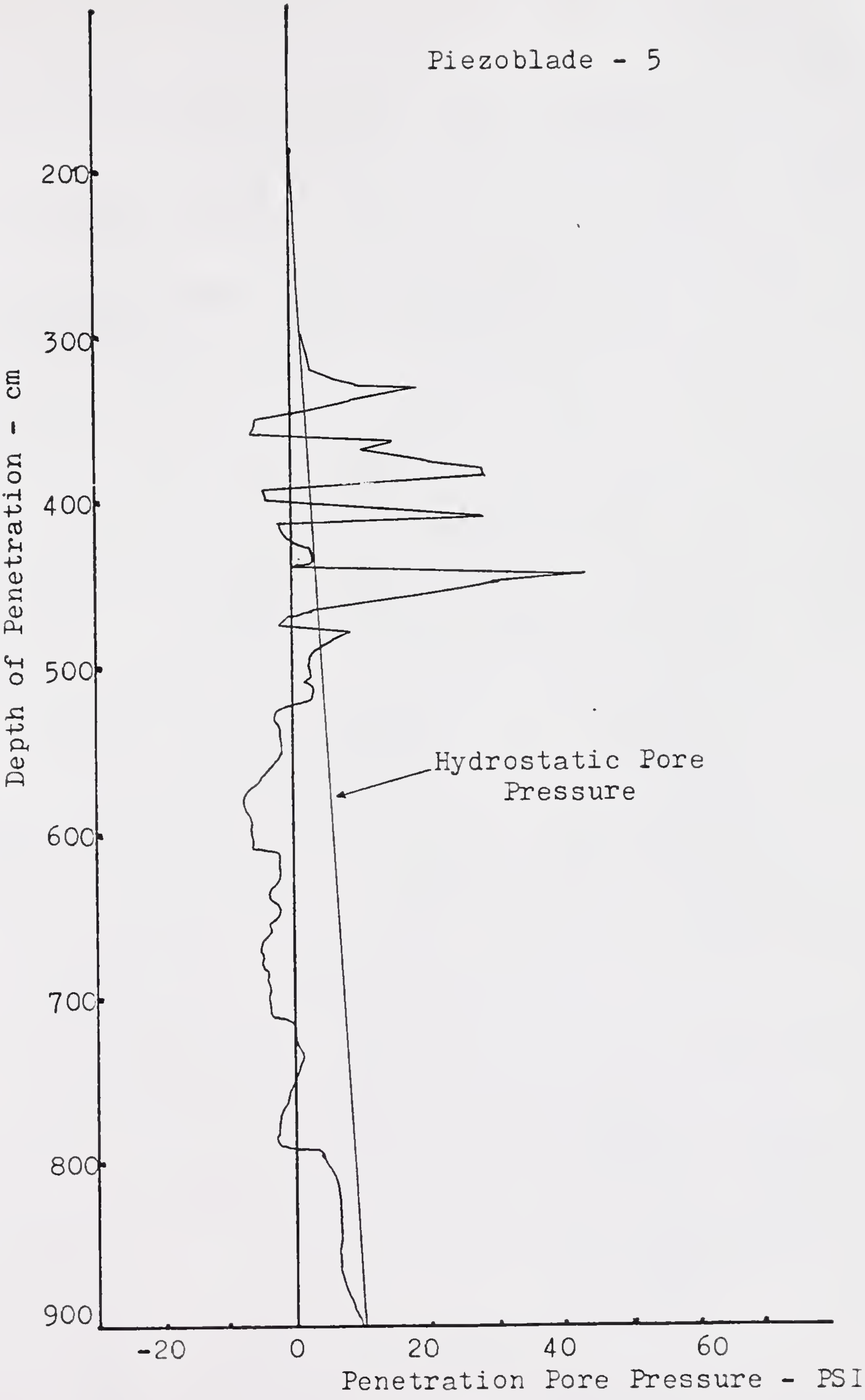


Figure A-5. Pore Pressure Profile from Piezoblade-5 (Continuous Penetration)

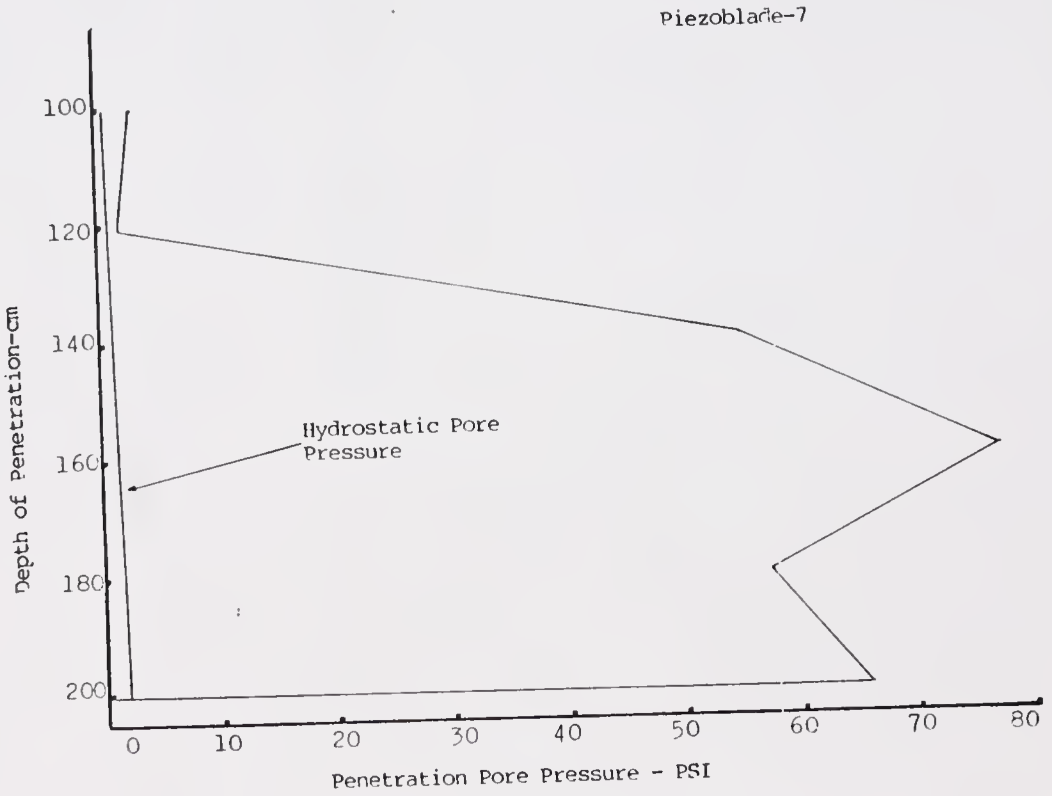


Figure A-7. Initial Pore Pressure Readings from Piezoblade-7 (Incremental Penetration)

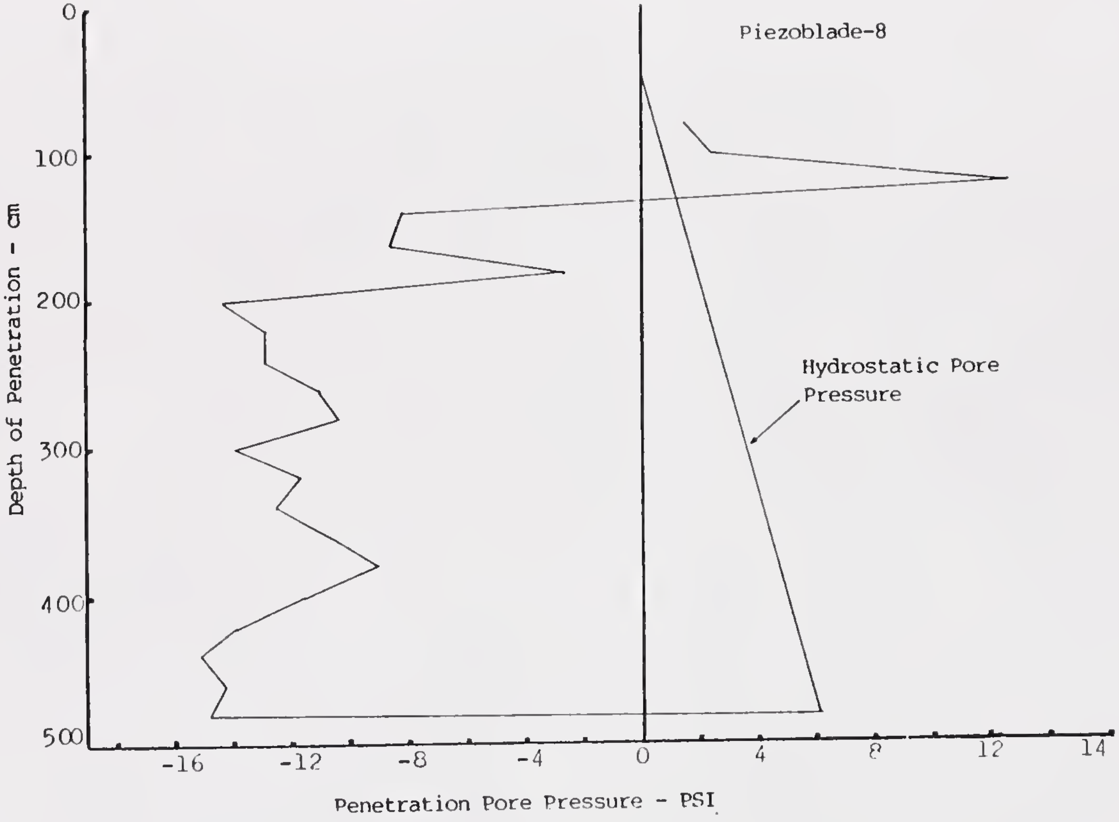


Figure A-8. Initial Pore Pressure Readings from Piezoblade-8 (Incremental Penetration)

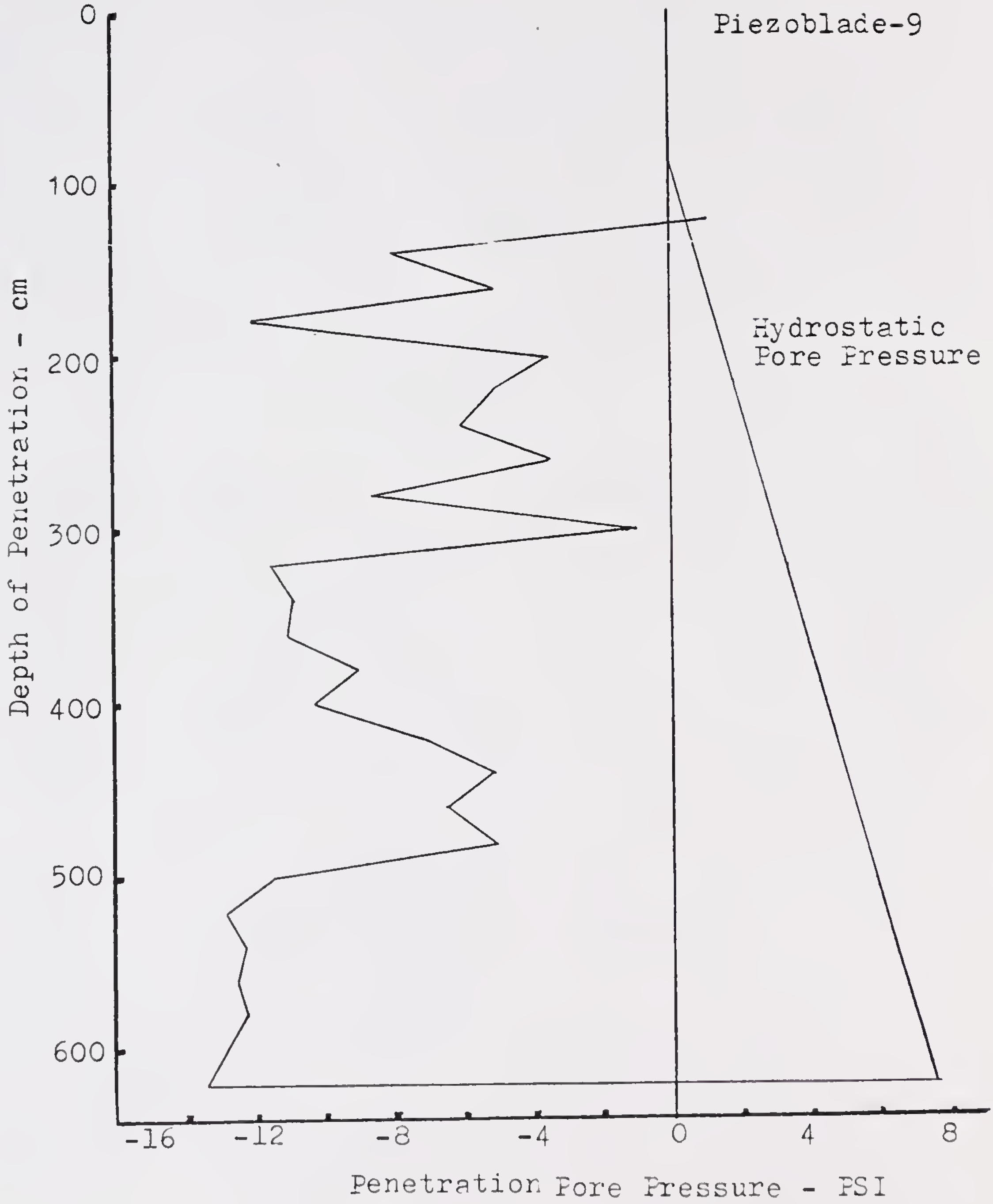


Figure A-9. Initial Pore Pressure Readings from Piezoblade-9 (Incremental Penetration)

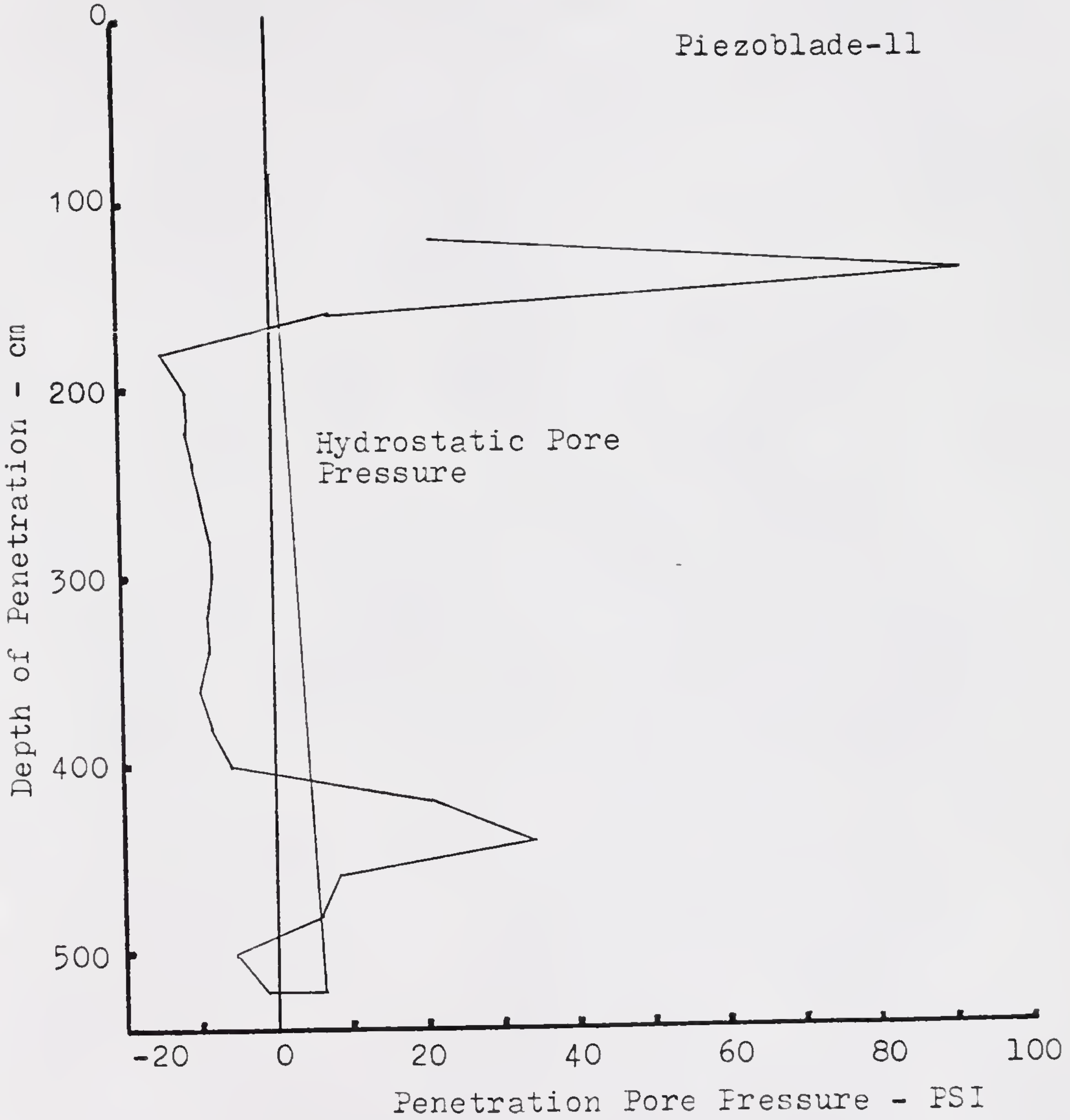


Figure A-10. Initial Pore Pressure Readings from Piezoblade-11 (Incremental Penetration)

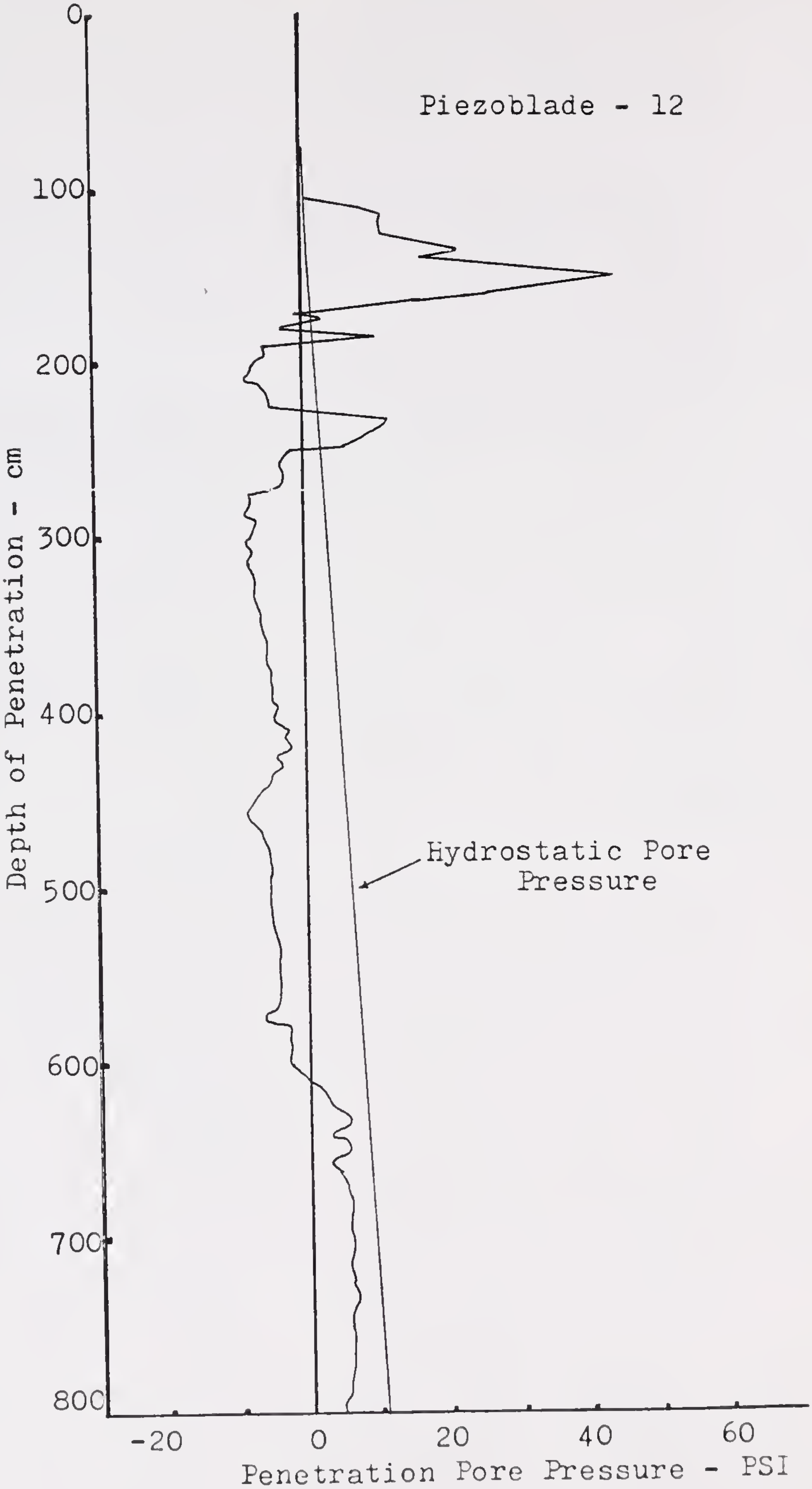


Figure A-11. Pore Pressure Profile from Piezoblade-12 (Continuous Penetration)

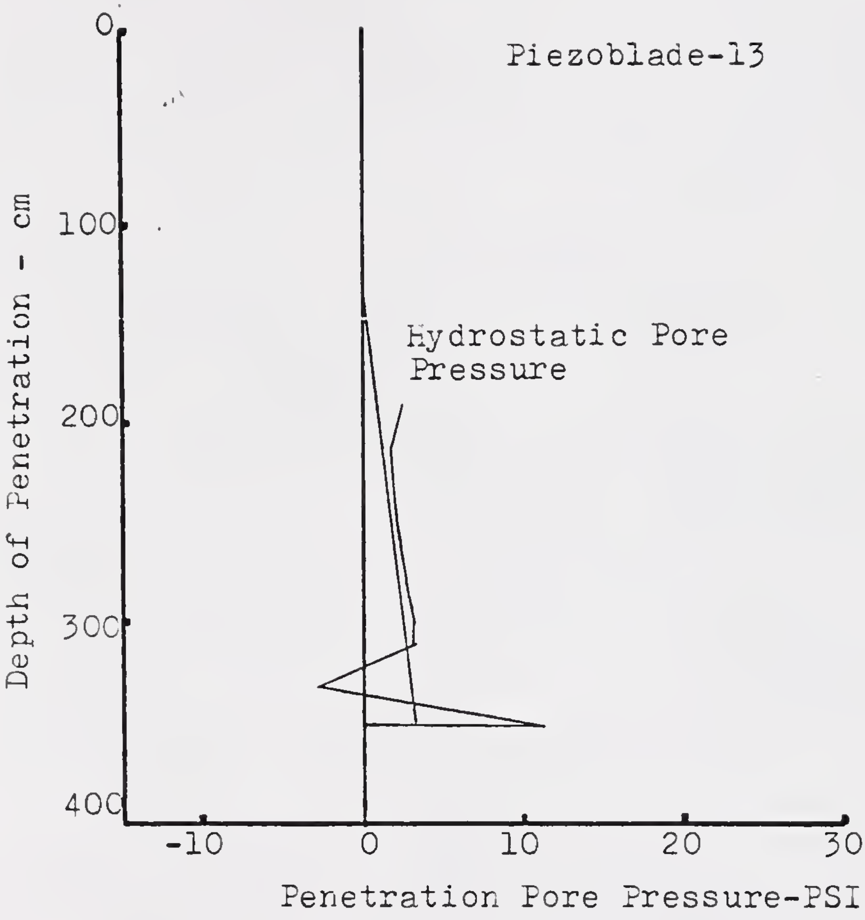
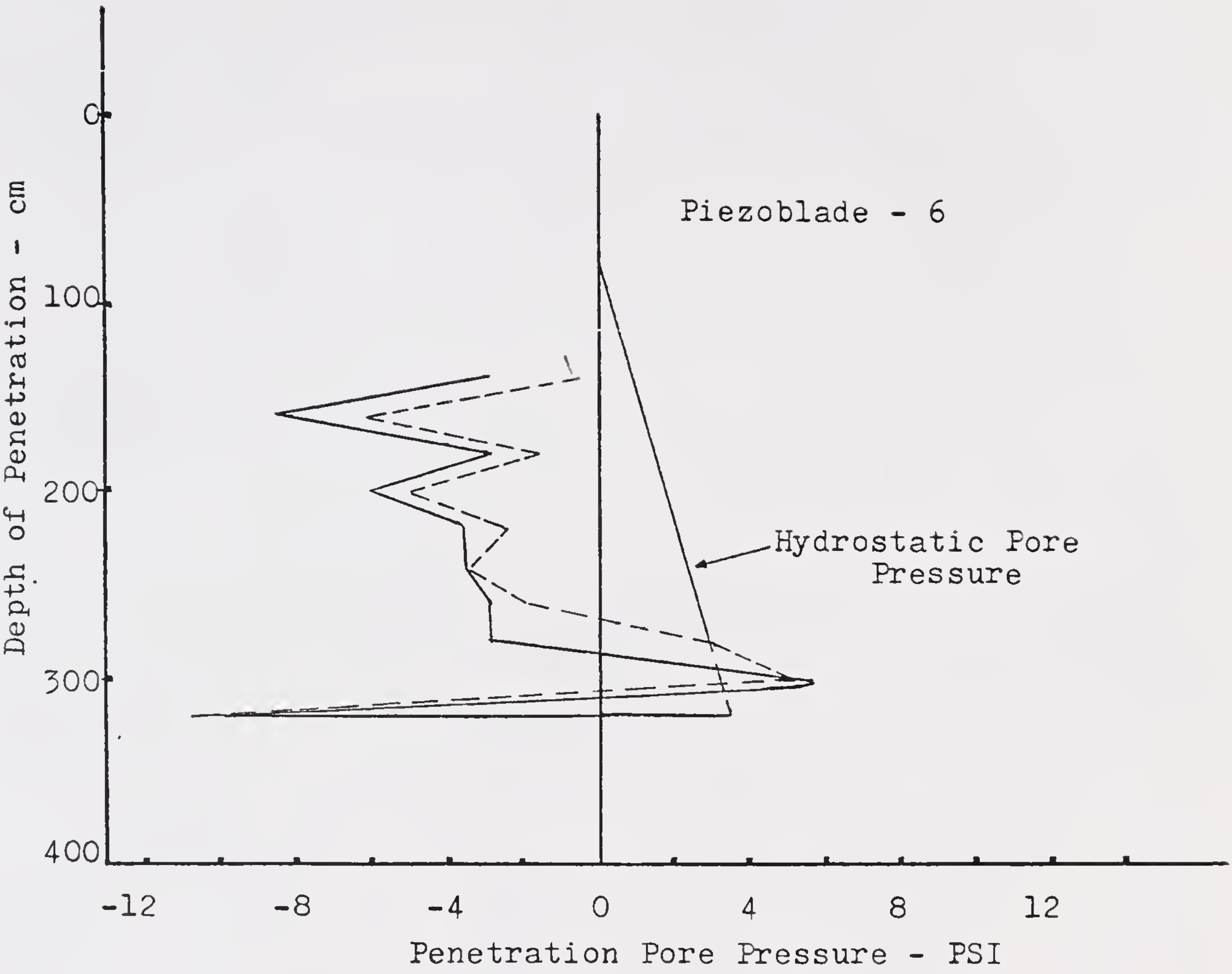


Figure A-12. Initial Pore Pressure Readings from Piezoblade-13 (Incremental Penetration)



—— Eight seconds dissipation

- - - Sixty seconds dissipation

Figure A-13. Dissipation of Excess Pore Pressures (Piezoblade stopped every 20 centimeters)

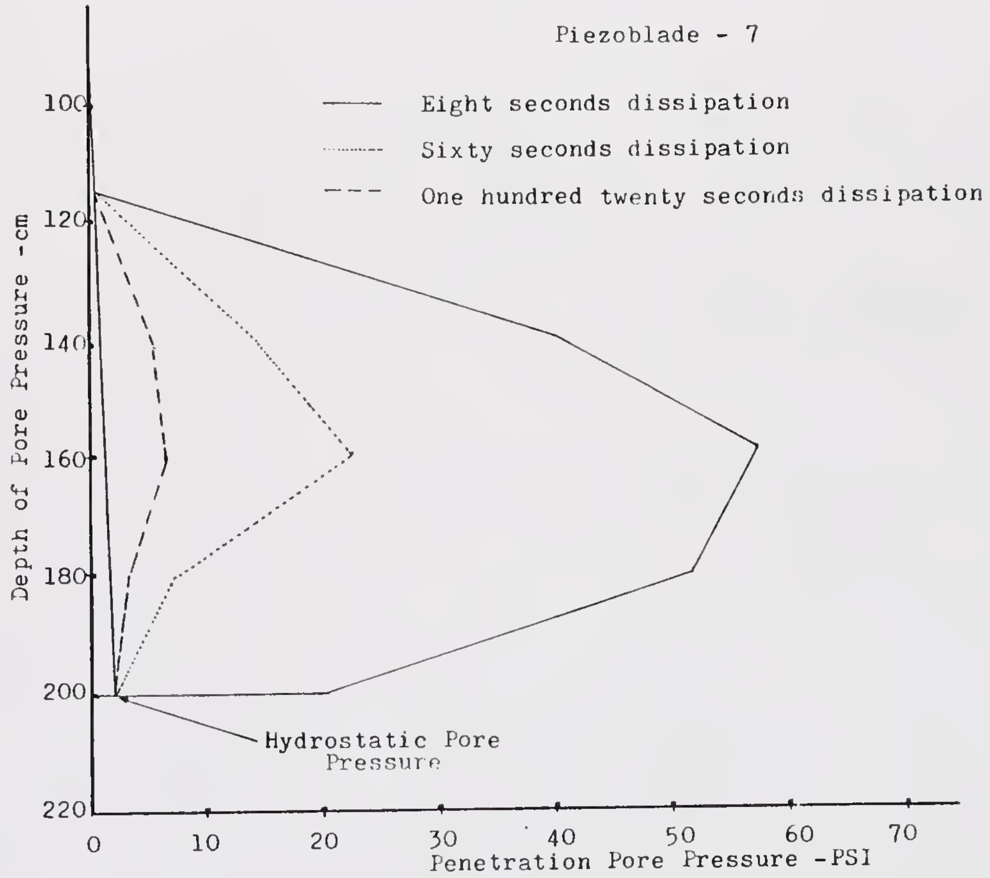


Figure A-14. Dissipation of Excess Pore Pressures (Piezoblade stopped every 20 centimeters)

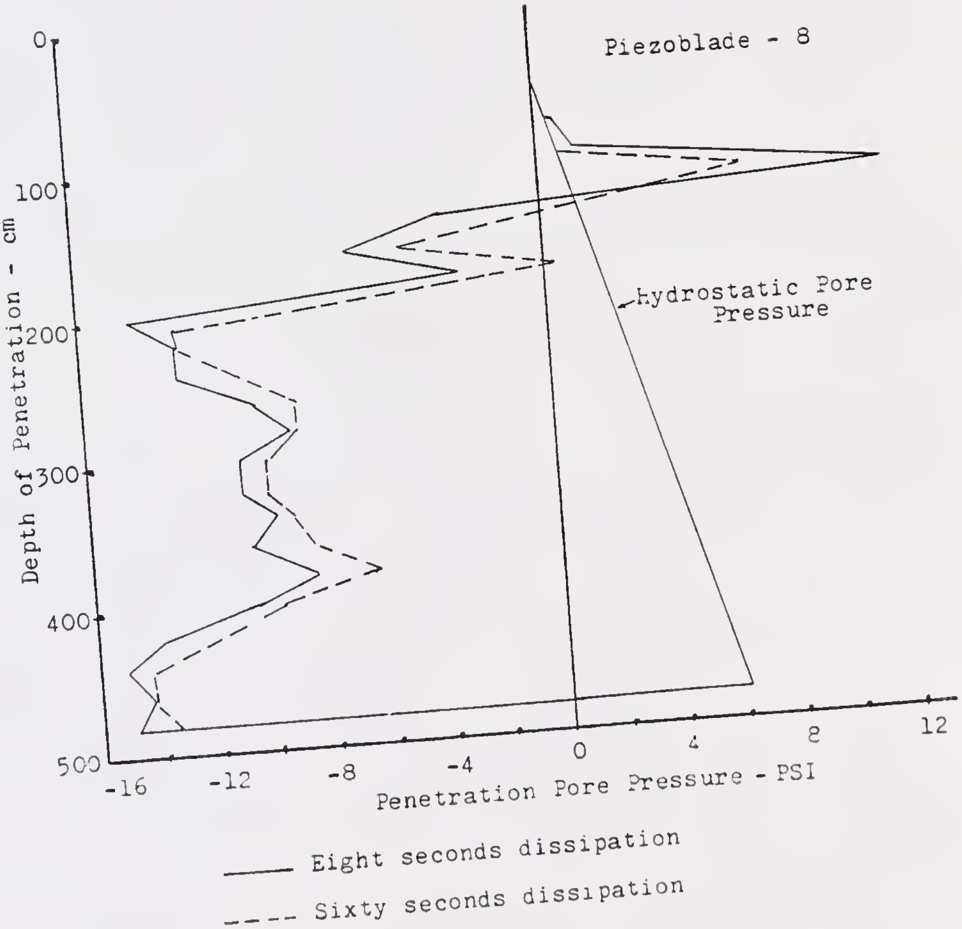


Figure A-15. Dissipation of Excess Pore Pressures (Piezoblade stopped every 20 centimeters)

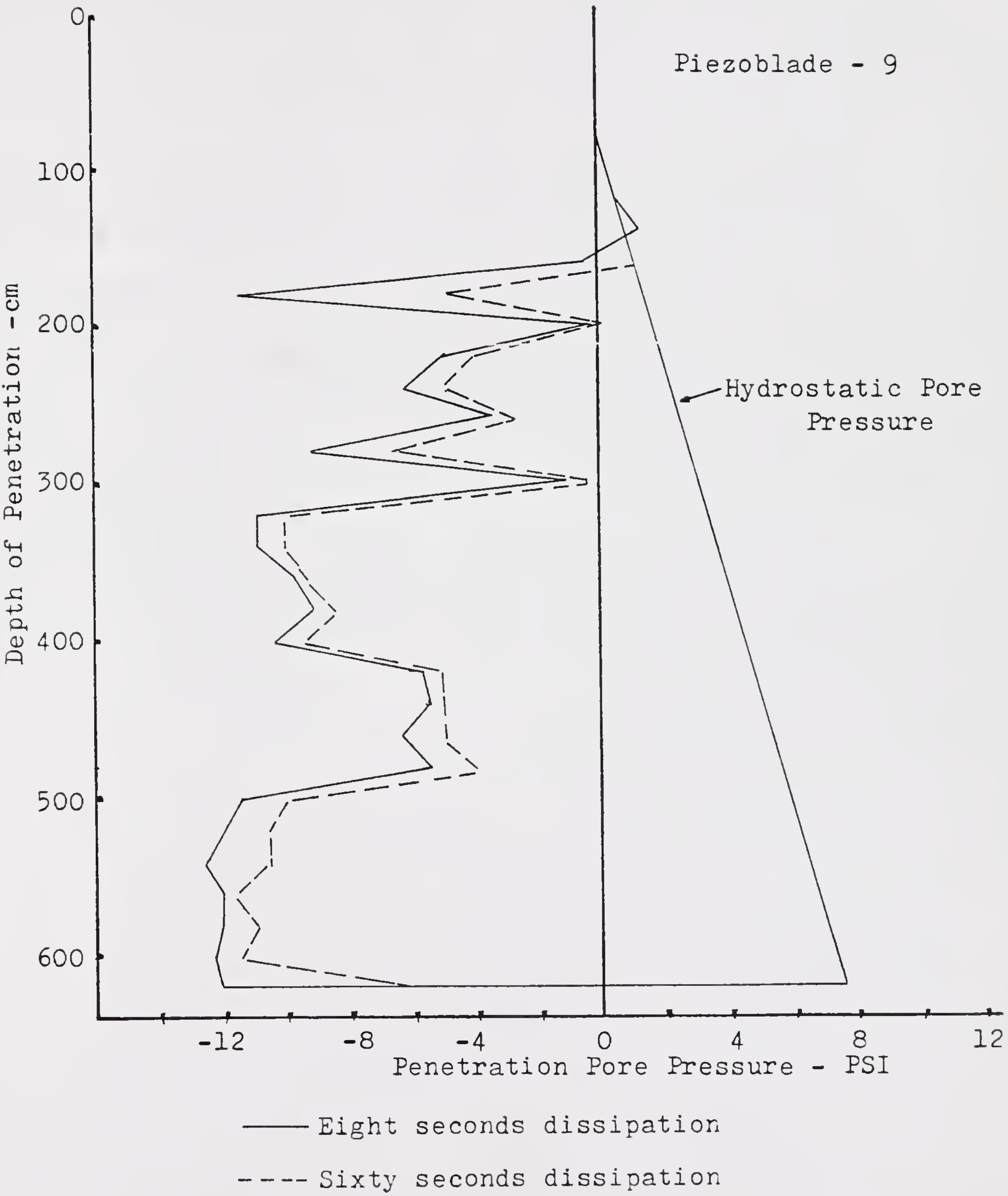


Figure A-16. Dissipation of Excess Pore Pressures (Piezoblade stopped every 20 centimeters)

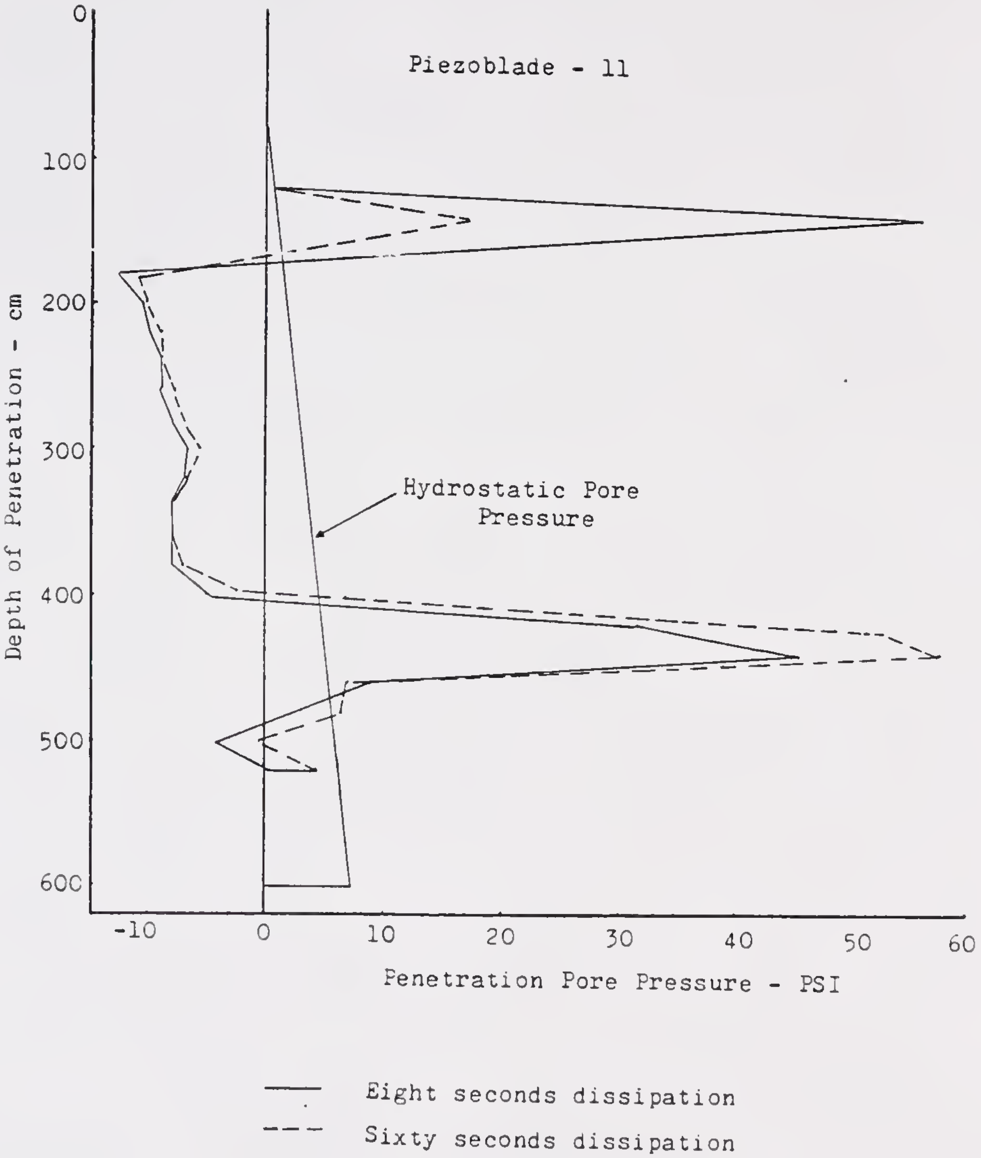


Figure A-17. Dissipation of Excess Pore Pressures (Piezoblade stopped every 20 centimeters)

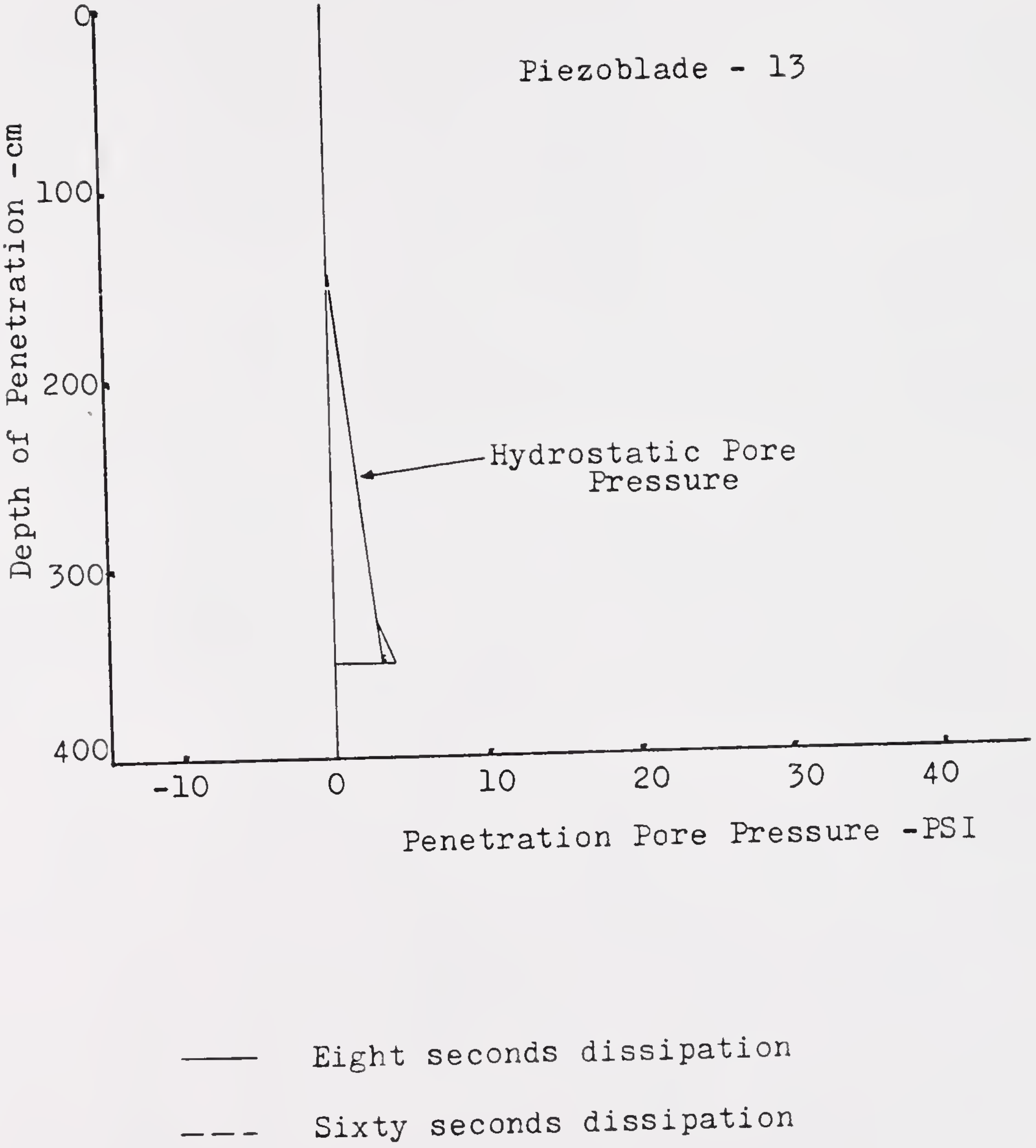


Figure A-18. Dissipation of Excess Pore Pressures (Piezoblade stopped every 20 centimeters)

APPENDIX B

RESULTS OF DILATOMETER TESTS

```
FILE NAME:     PIEZOBLADE RESEARCH
FILE NUMBER:   1

RECORD OF DILATOMETER TEST NO. D-1
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GED, MARCH 1980

     LOCATION: LAKE ALICE (NORTH)
     PERFORMED:  DATE:  1 JULY 1981
                 BY  :  ALIREZA BOGHRAT


     CALIBRATION INFORMATION:
     DA=  0.20 BARS      DB=  0.20 BARS      ZM=  0.20 BARS      ZW=  0.90 METERS


  Z     A     B      ED     UO    ID   GAMMA   SV      KD    OCR    PC    KO   CU     M       SOIL TYPE        DESCRIPTION
##### ##### ##### ###### ##### ##### ##### ###### ###### ##### ##### #### ##### ###### ############# ###################
 0.20  1.60  4.20   85.5  0.00  1.55  1.70  0.035  42.57 *****  8.37 4.22        317.0  SANDY SILT      LOW DENSITY
 0.40  0.80  5.60  170.9  0.00  7.97  1.70  0.069   6.41 27.15  1.87 1.65        401.6     SAND            LOOSE
 0.60  1.20  6.20  178.7  0.00  4.98  1.80  0.105   9.24 32.51  3.41 1.75        434.5     SAND         LOW RIGIDITY
 0.80  2.00  8.80  243.6  0.00  4.00  1.90  0.143  11.75 51.46  7.36 2.03        656.4     SAND       MEDIUM RIGIDITY
 1.00  3.00  7.40  155.4  0.01  1.51  1.80  0.169  16.51 43.85  7.42 2.49        443.3  SANDY SILT    MEDIUM DENSITY
 1.20  4.00 12.00  318.6  0.03  2.42  1.90  0.187  19.04 ***** 24.19 2.70        949.8  SILTY SAND   MEDIUM RIGIDITY



 1.40  6.60 13.40  442.9  0.05  2.00  2.00  0.207  28.89 ***** 59.40 3.42       1487.4  SILTY SAND          RIGID
 1.60  4.20 11.00  248.6  0.07  1.76  1.80  0.223  17.09 71.05 15.85 2.54        717.0  SANDY SILT    MEDIUM DENSITY
 1.80  4.80 11.80  256.4  0.09  1.58  1.80  0.239  18.33 59.51 14.22 2.64        755.7  SANDY SILT    MEDIUM DENSITY
 2.00  3.60 11.60  295.3  0.11  2.57  1.90  0.257  12.10 54.45 13.99 2.07        759.5  SILTY SAND   MEDIUM RIGIDITY
 2.20  5.00 14.85  367.1  0.13  2.26  2.00  0.277  15.88 91.45 25.33 2.43       1034.4  SILTY SAND          RIGID
 2.40  6.80 15.70  330.2  0.15  1.43  1.95  0.296  21.03 58.46 17.31 2.86       1014.3  SANDY SILT          DENSE
 2.60  5.80 13.20  271.9  0.17  1.39  1.95  0.315  16.76 37.49 11.81 2.51        779.6  SANDY SILT          DENSE
 2.80  7.80 15.40  279.7  0.19  1.04  1.95  0.334  21.71 41.26 13.78 2.91        867.2     SILT             DENSE
 3.00  5.60 13.40  287.5  0.21  1.55  1.95  0.353  14.22 36.40 12.85 2.28        781.4  SANDY SILT          DENSE
 3.20  4.00 10.60  240.9  0.23  1.88  1.90  0.371   9.33 28.04 10.40 1.76        586.4  SILTY SAND   MEDIUM RIGIDITY
 3.40  6.40 15.20  365.2  0.25  1.74  1.95  0.390  14.56 51.10 19.93 2.31       1000.5  SANDY SILT          DENSE
 3.60  6.20 14.80  318.6  0.27  1.36  1.95  0.409  13.50 33.90 13.86 2.21        850.9  SANDY SILT          DENSE
 3.80  4.80 15.40  332.2  0.29  3.76  2.00  0.429   8.92 30.38 13.03 1.71       1277.6     SAND             RIGID
 4.00  6.20 10.10  136.0  0.31  0.64  1.60  0.449  12.84 18.19  8.10 2.14  1.00  357.1  CLAYEY SILT   MEDIUM DENSITY
 4.20  8.40 14.60  225.3  0.33  0.73  1.95  0.464  16.77 27.59 12.80 2.51  1.46  646.0  CLAYEY SILT         DENSE
 4.40  9.70 16.50  248.6  0.35  0.74  1.95  0.483  18.70 32.68 15.79 2.67  1.74  737.3  CLAYEY SILT         DENSE
 4.60 12.00 29.60  629.4  0.37  1.57  2.10  0.505  21.43 76.59 38.68 2.89       1943.7  SANDY SILT        VERY DENSE
 4.80  9.80 29.40  707.1  0.39  2.25  2.15  0.528  16.10 93.92 49.59 2.45       2001.1  SILTY SAND        VERY RIGID
 5.00 11.20 27.40  613.8  0.41  1.66  2.10  0.550  18.18 66.71 36.69 2.63       1804.7  SANDY SILT        VERY DENSE
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FILE NAME:      PIEZOBLADE RESEARCH
FILE NUMBER:    2

RECORD OF DILATOMETER TEST NO. D-2
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GED, MARCH 1980

    LOCATION: LAKE ALICE (NORTH)
    PERFORMED:  DATE:  6 JULY 1981
                BY  :  ALIREZA BOGHRAT


    CALIBRATION INFORMATION:
    DA=  0.10 BARS     DB=  0.20 BARS     ZM=  0.20 BARS     ZW=  0.90 METERS


  Z     A      B      ED     UO    ID   GAMMA   SV      KD    OCR    PC    KO   CU      M      SOIL TYPE       DESCRIPTION
##### ##### ##### ###### ##### ##### ##### ###### ###### ##### ##### #### ##### ###### ############ ##################
 0.20  2.20 11.80  361.3  0.00  5.97  1.90  0.035  46.71 ***** 25.15 4.43       1387.0     SAND      MEDIUM RIGIDITY
 0.40  2.00 10.40  314.7  0.00  5.69  1.90  0.073  20.48 ***** 10.86 2.82        982.6     SAND      MEDIUM RIGIDITY
 0.60  2.20 11.40  345.8  0.00  5.65  1.90  0.111  14.91 51.12  9.00 2.34        984.4     SAND      MEDIUM RIGIDITY
 0.80  1.80  9.80  299.1  0.00  6.15  1.80  0.147   8.95 30.53  4.49 1.71        718.9     SAND       LOW RIGIDITY
 1.00  3.40 11.00  283.6  0.01  2.62  1.90  0.175  16.71 ***** 17.66 2.51        812.3  SILTY SAND  MEDIUM RIGIDITY
 1.20  6.20 19.60  509.9  0.03  2.54  2.00  0.195  27.77 ***** 51.89 3.34       1691.0  SILTY SAND       RIGID



 1.40 10.20 40.00 1146.1  0.05  3.61  2.15  0.218  39.33 ***** ***** 4.04       4228.5     SAND         VERY RIGID
 1.60 10.80 34.00  889.7  0.07  2.54  2.15  0.241  39.36 ***** ***** 4.04       3236.3  SILTY SAND     VERY RIGID
 1.80 11.60 34.40  874.1  0.09  2.30  2.15  0.264  38.96 ***** ***** 4.02       3171.7  SILTY SAND     VERY RIGID
 2.00  7.20 24.20  648.8  0.11  2.85  2.00  0.284  21.67 ***** 47.07 2.91       2010.4  SILTY SAND        RIGID
 2.20  8.10 26.20  691.5  0.13  2.68  2.00  0.304  22.96 ***** 56.25 3.00       2178.9  SILTY SAND        RIGID
 2.40  5.00 15.60  400.2  0.15  2.55  2.00  0.324  13.07 63.09 20.44 2.17       1057.2  SILTY SAND        RIGID
 2.60  6.20 23.10  644.9  0.17  3.42  2.00  0.344  14.83 80.25 27.61 2.34       1832.8     SAND           RIGID
 2.80  7.20 15.80  322.5  0.19  1.34  1.95  0.363  17.09 38.44 13.96 2.61        943.4  SANDY SILT        DENSE
 3.00  6.10 13.60  279.7  0.21  1.39  1.95  0.382  14.21 28.60 10.93 2.28        760.2  SANDY SILT        DENSE
 3.20  6.80 14.40  283.6  0.23  1.26  1.95  0.401  15.22 25.66 10.37 2.37        788.3  SANDY SILT        DENSE
 3.40  8.60 20.40  446.9  0.25  1.57  1.95  0.420  18.27 58.34 24.50 2.64       1315.6  SANDY SILT        DENSE
 3.60  9.40 16.80  273.8  0.27  0.86  1.95  0.439  15.76 33.63 15.04 2.76  1.69  831.7     SILT           DENSE
 3.80  9.40 13.80  190.4  0.29  0.66  1.95  0.458  16.95 28.06 12.35 2.53  1.46  547.7  CLAYEY SILT       DENSE
 4.00 11.00 20.90  369.1  0.31  0.99  2.10  0.480  21.07 39.39 18.91 2.86  2.00 1134.3     SILT        VERY DENSE
 4.20 16.20 40.00  913.9  0.33  1.69  2.10  0.502  29.07 ***** 81.22 3.43       3071.3  SANDY SILT     VERY DENSE
 4.40 18.00 40.00  843.0  0.35  1.38  2.10  0.524  31.42 ***** 54.35 3.58       2895.2  SANDY SILT     VERY DENSE
 4.60 18.40 40.00  827.5  0.37  1.33  2.10  0.546  30.99 90.51 49.42 3.54       2829.0  SANDY SILT     VERY DENSE
 4.80 11.80 24.40  477.9  0.39  1.21  2.10  0.568  18.83 33.46 19.01 2.68       1420.0  SANDY SILT     VERY DENSE
 5.00 11.20 17.20  299.1  0.41  0.78  1.95  0.587  17.56 29.63 17.39 2.58  1.95  870.0  CLAYEY SILT       DENSE
 5.20 10.80 18.40  283.6  0.43  0.77  1.95  0.606  16.34 26.59 16.06 2.47  1.84  806.5  CLAYEY SILT       DENSE
 5.40  8.20 14.60  213.7  0.45  0.72  1.95  0.625  12.76 18.61 11.26 2.14  1.39  559.9  CLAYEY SILT       DENSE
 5.60  9.40 20.60  423.5  0.47  1.38  1.95  0.644  12.86 23.88 15.38 2.15       1112.7  SANDY SILT        DENSE
 5.80  8.00 14.40  237.0  0.49  0.90  1.95  0.663  10.72 13.72  9.09 1.92  1.19  583.6     SILT           DENSE
 6.00  9.20 18.60  353.5  0.51  1.17  1.95  0.682  11.93 16.21 11.06 2.05        904.8     SILT           DENSE



 6.20  5.40 14.20  407.9  0.53  2.60  2.00  0.702   6.05 14.47 10.16 1.33        836.2  SILTY SAND        RIGID
 6.40  8.60 19.00  353.5  0.55  1.27  1.95  0.721  10.40 14.51 10.46 1.88        660.9  SANDY SILT        DENSE
 6.60  6.80 13.80  260.3  0.57  1.21  1.95  0.740   7.83  8.56  6.33 1.57        598.1  SANDY SILT        DENSE
 6.80  4.50 14.80  373.0  0.59  2.70  1.90  0.758   4.92  5.76  7.40 1.15        697.6  SILTY SAND  MEDIUM RIGIDITY
 7.00  7.80 14.60  252.5  0.61  1.01  1.95  0.777   8.71  9.92  7.71 1.69        596.4     SILT           DENSE
 7.20  7.40 12.10  170.9  0.63  0.72  1.95  0.796   8.10  8.67  7.06 1.61  1.01  390.8  CLAYEY SILT       DENSE
 7.40  5.90  9.90  143.7  0.65  0.78  1.80  0.812   6.11  5.72  4.64 1.34  0.72  287.7  CLAYEY SILT  MEDIUM DENSITY
 7.60  6.20 13.20  260.3  0.67  1.38  1.95  0.831   6.13  7.08  5.89 1.34        526.7  SANDY SILT        DENSE
 7.80  6.20 10.60  159.3  0.69  0.83  1.80  0.847   6.15  5.76  4.88 1.34  0.76  319.9     SILT      MEDIUM DENSITY
 8.00  4.10 12.20  303.0  0.71  2.82  1.90  0.865   3.35  4.69  4.06 0.86        466.1  SILTY SAND  MEDIUM RIGIDITY
 6.20  3.90  9.40  202.0  0.73  1.94  1.90  0.883   3.18  4.03  3.56 0.82        288.2  SILTY SAND  MEDIUM RIGIDITY
 8.40  3.40  8.40  182.6  0.75  2.13  1.90  0.901   2.57  2.82  2.54 0.69        226.9  SILTY SAND  MEDIUM RIGIDITY
 6.60  2.80  7.20  159.3  0.77  2.50  1.80  0.917   1.88  1.56  1.43 0.51        158.3  SILTY SAND   LOW RIGIDITY
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FILE NAME:      PIEZOBLADE RESEARCH
FILE NUMBER:    3

RECORD OF DILATOMETER TEST NO. D-3
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GED, MARCH 1980

     LOCATION: LAKE ALICE (EAST)
     PERFORMED:  DATE:  9 JULY 1981
                 BY  :  ALIREZA BOGHRAT


     CALIBRATION INFORMATION:
     DA=  0.10 BARS     DB=  0.20 BARS     ZM=  0.20 BARS     ZW=  0.45 METERS


  Z     A     B     ED     UO    ID   GAMMA   SV     KD    CCR   PC    KO   CU      M      SOIL TYPE       DESCRIPTION
##### ##### ##### ###### ##### ##### ##### ###### ###### ##### ##### #### ##### ###### ############ ##################
 0.20  1.50 10.80  349.6  0.00  9.95  1.80  0.035  27.14 *****  8.92 3.30        1177.4     SAND        LOW RIGIDITY
 0.40  3.50 18.00  551.7  0.00  5.54  1.90  0.073  36.85 ***** 33.34 3.90        2004.1     SAND       MEDIUM RIGIDITY
 0.60  4.50 16.40  450.7  0.02  3.20  2.00  0.098  38.83 ***** 49.46 4.01        1657.6  SILTY SAND        RIGID
 0.80  2.40  5.80  120.4  0.04  1.54  1.70  0.112  18.84 58.41  6.54 2.68         357.9  SANDY SILT     LOW DENSITY
 1.00  5.80 16.00  462.3  0.05  2.47  2.00  0.132  38.26 ***** 64.77 3.98        1669.9  SILTY SAND        RIGID
 1.20  6.80 28.10  815.8  0.07  3.96  2.00  0.152  36.66 ***** 68.81 3.89        2960.6     SAND           RIGID



 1.40  8.10 17.10  338.0  0.09  1.22  1.95  0.171  43.68 ***** 21.99 4.28        1261.4  SANDY SILT        DENSE
 1.60  9.80 21.00  423.5  0.11  1.27  1.95  0.190  47.58 ***** 30.53 4.48        1613.0  SANDY SILT        DENSE
 1.80  8.20 16.40  306.9  0.13  1.10  1.95  0.209  36.22 91.70 19.16 3.87        1093.4     SILT           DENSE
 2.00 10.80 23.10  466.2  0.15  1.27  2.10  0.231  43.05 ***** 31.73 4.24        1733.7  SANDY SILT     VERY DENSE
 2.20 10.00 24.20  540.0  0.17  1.62  2.10  0.253  35.69 ***** 50.90 3.84        1916.7  SANDY SILT     VERY DENSE
 2.40  9.00 16.80  291.4  0.19  0.95  1.95  0.272  30.63 70.58 19.20 3.53  1.81  993.9     SILT           DENSE
 2.60 10.40 19.80  353.5  0.21  0.99  1.95  0.291  33.09 79.65 23.18 3.68  2.14 1230.7     SILT           DENSE
 2.80  8.00 14.80  252.5  0.23  0.93  1.95  0.310  23.68 47.24 14.65 3.06  1.50  802.7     SILT           DENSE
 3.00  9.50 26.80  660.4  0.25  2.15  2.15  0.333  24.91 ***** 72.00 3.15        2129.6  SILTY SAND     VERY RIGID
 3.20  8.40 19.20  407.9  0.27  1.47  1.95  0.352  21.31 63.59 22.39 2.98        1257.8  SANDY SILT        DENSE
 3.40  8.20 16.20  299.1  0.29  1.09  1.95  0.371  20.00 36.31 13.47 2.76         905.3     SILT           DENSE
 3.60 10.10 17.20  264.2  0.31  0.76  1.95  0.390  23.96 48.13 18.77 3.08  1.91  842.6  CLAYEY SILT       DENSE
 3.80 11.80 19.40  283.6  0.33  0.70  1.95  0.409  26.89 57.63 23.57 3.28  2.32  934.1  CLAYEY SILT       DENSE
 4.00  7.80 19.60  446.8  0.35  1.78  1.95  0.428  15.82 63.93 27.32 2.43        1257.3  SANDY SILT        DENSE
 4.20  7.50 21.90  547.8  0.37  2.34  2.00  0.448  14.11 72.93 32.70 2.27        1494.9  SILTY SAND        RIGID
 4.40  7.20 15.10  295.3  0.39  1.26  1.95  0.467  13.54 21.28 10.13 2.21         789.5  SANDY SILT        DENSE
 4.60  5.30 12.20  256.4  0.41  1.56  1.80  0.481   9.22 17.53  8.47 1.75         621.1  SANDY SILT    MEDIUM DENSITY
 4.80  6.00 12.80  252.5  0.43  1.33  1.95  0.502  10.24 15.25  7.65 1.87         611.4  SANDY SILT        DENSE
 5.00  7.20 11.80  167.1  0.45  0.70  1.95  0.516  12.46 17.36  8.96 2.10  1.12  434.1  CLAYEY SILT       DENSE
 5.20  6.80 10.80  143.7  0.47  0.64  1.80  0.532  11.35 15.01  7.99 1.99  1.03  361.5  CLAYEY SILT   MEDIUM DENSITY
 5.40  7.80 11.00  112.7  0.49  0.43  1.90  0.550  12.84 18.18 10.00 2.14  1.24  310.5  SILTY CLAY  HIGH CONSISTENCY
 5.60  7.10 10.80  132.1  0.51  0.57  1.80  0.566  11.16 14.61  8.27 1.97  1.07  345.0  SILTY CLAY MEDIUM CONSISTENCY
 5.60  6.80 11.80  182.6  0.53  0.83  1.95  0.585  10.14 12.58  7.36 1.85  0.93  440.5     SILT           DENSE
 6.00  7.20 11.50  155.4  0.55  0.66  1.95  0.604  10.50 13.30  8.03 1.90  1.06  379.9  CLAYEY SILT       DENSE



 6.20  7.30 11.40  147.6  0.57  0.62  1.95  0.623  10.33 12.95  8.07 1.88  1.07  358.6  CLAYEY SILT       DENSE
 6.40  6.80 11.50  170.9  0.59  0.79  1.95  0.642   9.17 10.75  6.90 1.74  0.95  412.3  CLAYEY SILT       DENSE
 6.60  6.10 10.00  139.9  0.61  0.73  1.80  0.658   7.91  8.54  5.62 1.58  0.81  316.3  CLAYEY SILT   MEDIUM DENSITY
 6.80  4.80 14.50  365.2  0.63  2.75  1.90  0.676   5.32 11.32  7.65 1.21         708.8  SILTY SAND   MEDIUM RIGIDITY
 7.00  5.50  9.10  128.2  0.65  0.76  1.80  0.692   6.62  6.47  4.48 1.41  0.68  266.8  CLAYEY SILT   MEDIUM DENSITY
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FILE NAME:      PIEZOBLADE RESEARCH
FILE NUMBER:    4

RECORD OF DILATOMETER TEST NO. D-4
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GED, MARCH 1980

     LOCATION: LAKE ALICE (WEST)
     PERFORMED:  DATE:  20 JULY 1981
                 BY  :  ALIREZA BOGHRAT


     CALIBRATION INFORMATION:
     DA=  0.20 BARS     DB=  0.20 BARS     ZM=  0.20 BARS     ZW=  1.50 METERS


  Z     A     B     ED     U0    ID   GAMMA   SV      KD   OCR    PC   K0    CU     M     SOIL TYPE      DESCRIPTION
##### ##### ##### ###### ##### ##### ##### ###### ###### ##### ##### #### ##### ###### ############# ####################
 0.20  1.20  6.30  202.0  0.00  5.81  1.80  0.035  26.86 *****  8.74 3.28        678.4     SAND        LOW RIGIDITY
 0.40  0.60  9.00  303.0  0.00 19.98  1.70  0.069   5.94 14.00  0.97 1.31        620.6     SAND            LOOSE
 0.60  0.80  6.10  190.4  0.00  9.27  1.70  0.103   5.39 11.61  1.20 1.22        373.7     SAND            LOOSE
 0.80  0.60  4.60  139.9  0.00  9.00  1.70  0.137   3.07  3.95  0.54 0.80        206.0     SAND            LOOSE
 1.00  0.40  3.10   89.4  0.00  8.47  1.70  0.171   1.67  1.23  0.21 0.45         94.3     SAND            LOOSE
 1.20  0.40  3.00   85.5  0.00  7.97  1.70  0.205   1.41  0.90  0.19 0.37         72.6     SAND            LOOSE



 1.40  0.40  3.10   89.4  0.00  8.47  1.70  0.239   1.19  0.65  0.16 0.30         76.0     SAND            LOOSE
 1.60  0.30  2.80   81.6  0.01 11.92  1.70  0.263   0.70  0.24  0.06 0.10         69.3     SAND            LOOSE
 1.80  0.40  3.30   97.1  0.03 10.71  1.70  0.277   0.68  0.37  0.10 0.18         82.6     SAND            LOOSE
 2.00  1.10  4.10  101.0  0.05  2.97  1.70  0.291   3.16  4.19  1.22 0.82        151.2  SILTY SAND        LOOSE
 2.20  1.10  4.80  128.2  0.07  4.01  1.80  0.307   2.82  3.37  1.03 0.74        179.5     SAND        LOW RIGIDITY
 2.40  1.30  5.60  151.5  0.09  4.03  1.80  0.323   3.14  4.15  1.34 0.82        226.4     SAND        LOW RIGIDITY
 2.60  1.30  6.20  174.8  0.11  4.90  1.80  0.339   2.85  3.43  1.16 0.75        246.3     SAND        LOW RIGIDITY
 2.80  1.80  8.30  237.0  0.13  4.69  1.80  0.355   3.85  6.09  2.16 0.96        395.7     SAND        LOW RIGIDITY
 3.00  3.40 12.60  341.9  0.15  3.29  1.90  0.373   7.53 22.02  8.21 1.54        770.6  SILTY SAND   MEDIUM RIGIDITY
 3.20  2.80 10.80  295.3  0.17  3.55  1.90  0.391   5.75 13.16  5.15 1.28        596.4     SAND      MEDIUM RIGIDITY
 3.40  2.70  6.20  120.4  0.19  1.38  1.70  0.405   5.81  6.50  2.63 1.29        237.5  SANDY SILT     LOW DENSITY
 3.60  2.20  6.10  136.0  0.21  2.02  1.80  0.421   4.31  7.58  3.19 1.04        233.3  SILTY SAND     LOW RIGIDITY
 3.80  2.10  5.50  116.5  0.23  1.83  1.80  0.437   3.94  5.36  2.34 0.97        188.7  SILTY SAND     LOW RIGIDITY
 4.00  1.70  8.40  244.8  0.25  5.33  1.80  0.453   2.51  2.69  1.22 0.67        317.6     SAND        LOW RIGIDITY
 4.20  3.10  7.90  170.9  0.27  1.77  1.80  0.469   5.57  9.43  4.42 1.25        332.3  SANDY SILT   MEDIUM DENSITY
 4.40  2.50  6.70  147.6  0.29  1.93  1.80  0.485   4.16  6.92  3.35 1.02        248.1  SILTY SAND     LOW RIGIDITY
 4.60  3.70  8.60  174.8  0.31  1.49  1.80  0.501   6.32  8.50  4.26 1.37        359.4  SANDY SILT   MEDIUM DENSITY
 4.80  3.60  8.00  186.5  0.33  1.66  1.80  0.517   5.86  9.13  4.72 1.30        371.0  SANDY SILT   MEDIUM DENSITY
 5.00  4.70 11.20  237.0  0.35  1.58  1.80  0.533   7.59 13.00  6.93 1.54        529.6  SANDY SILT   MEDIUM DENSITY
 5.20  3.90  7.50  124.3  0.37  1.00  1.80  0.549   6.14  5.75  3.16 1.34  0.49  250.2     SILT      MEDIUM DENSITY
 5.40  4.60  9.10  159.3  0.39  1.07  1.80  0.565   7.09  7.20  4.07 1.47        343.7     SILT      MEDIUM DENSITY
 5.60  5.10  9.70  163.2  0.41  0.98  1.80  0.581   7.71  8.21  4.77 1.56  0.69  365.5     SILT      MEDIUM DENSITY
 5.80  4.40  7.30   97.1  0.43  0.68  1.80  0.597   6.44  6.20  3.70 1.38  0.57  199.2  CLAYEY SILT  MEDIUM DENSITY
 6.00  4.00  8.10  143.7  0.45  1.15  1.80  0.613   5.49  4.83  2.96 1.24        274.1     SILT      MEDIUM DENSITY



 6.20  4.90 10.30  194.2  0.47  1.26  1.80  0.629   6.65  6.96  4.38 1.41        407.7  SANDY SILT   MEDIUM DENSITY
 6.40  3.50  6.60  104.9  0.49  0.99  1.80  0.645   4.46  3.49  2.25 1.07  0.39  177.5     SILT      MEDIUM DENSITY
 6.60  3.50  6.30   81.6  0.51  0.69  1.70  0.659   4.83  3.96  2.61 1.13  0.44  143.5  CLAYEY SILT   LOW DENSITY
 6.80  4.50  8.90  155.4  0.53  1.11  1.80  0.675   5.54  4.96  3.35 1.26        298.8     SILT      MEDIUM DENSITY
 7.00  3.80  7.60  132.1  0.55  1.16  1.80  0.691   4.46  3.49  2.41 1.07        224.7     SILT      MEDIUM DENSITY
 7.20  3.70  7.00  112.7  0.57  1.02  1.80  0.707   4.22  3.21  2.27 1.03        184.8     SILT      MEDIUM DENSITY
 7.40  2.50  7.80  190.4  0.59  3.09  1.80  0.723   2.30  2.29  1.66 0.62        233.1  SILTY SAND     LOW RIGIDITY
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FILE NAME:     PIEZOBLADE RESEARCH
FILE NUMBER:   5

RECORD OF DILATOMETER TEST NO. D-5
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GED, MARCH 1980

     LOCATION: LAKE ALICE (WEST)
     PERFORMED:  DATE:  20 JULY 1981
                 BY  :  ALIREZA BOGHRAT


     CALIBRATION INFORMATION:
     DA=  0.20 BARS     DB=  0.20 BARS     ZM=  0.20 BARS     ZW=  1.90 METERS

    Z     A      B      ED    UO    ID  GAMMA     SV     KD    OCR     PC    KO    CU       M  SOIL TYPE      DESCRIPTION
##### ##### ###### ####### ##### ##### ###### ###### ###### ###### ###### ##### ##### ####### ############## ####################
 0.20  1.00   5.10   143.7  0.00  4.77   1.50  0.035  23.29  *****   6.65  3.03         454.9  SAND           LOW RIGIDITY
 0.40  2.60  12.50   369.1  0.00  4.69   1.90  0.073  29.11  *****  21.25  3.43        1265.2  SAND           MEDIUM RIGIDITY
 0.60  1.20   7.90   244.8  0.00  7.47   1.90  0.109   8.12  25.41   2.77  1.61         587.6  SAND           LOW RIGIDITY
 0.80  0.90   5.20   155.4  0.00  7.00   1.70  0.143   4.20   7.20   1.03  1.02         271.3  SAND           LOOSE
 1.00  0.50   3.50   101.0  0.00  7.38   1.70  0.177   2.09   1.93   0.34  0.57         115.2  SAND           LOOSE
 1.20  0.50   3.40    97.1  0.00  7.00   1.70  0.211   1.78   1.40   0.29  0.46          97.1  SAND           LOOSE


 1.40  0.90   3.90   101.0  0.00  3.55   1.70  0.245   3.14   4.15   1.02  0.82         151.0  SAND           LOOSE
 1.60  0.80   3.40    85.5  0.00  3.35   1.70  0.279   2.47   2.62   0.73  0.66         110.0  SAND           LOOSE
 1.80  0.70   3.50    93.2  0.00  4.34   1.70  0.313   1.85   1.51   0.47  0.50          96.6  SAND           LOOSE
 2.00  0.90   4.10   108.8  0.01  3.92   1.70  0.337   2.23   2.14   0.72  0.60         130.0  SAND           LOOSE
 2.20  1.20   5.60   151.5  0.03  3.81   1.80  0.353   3.05   3.90   1.38  0.79         222.3  SAND           LOW RIGIDITY
 2.40  2.20   7.80   202.0  0.05  2.89   1.90  0.371   5.09  10.43   3.87  1.18         385.7  SILTY SAND     MEDIUM RIGIDITY
 2.60  2.30   9.20   252.5  0.07  3.58   1.90  0.389   4.90   9.67   3.76  1.14         474.7  SAND           MEDIUM RIGIDITY
 2.80  2.80  11.00   303.0  0.09  3.53   1.90  0.407   5.70  12.93   5.26  1.27         609.6  SAND           MEDIUM RIGIDITY
 3.00  3.20  12.40   341.9  0.11  3.49   1.90  0.425   6.24  15.33   6.52  1.35         714.4  SAND           MEDIUM RIGIDITY
 3.20  3.80  13.60   365.2  0.13  3.08   1.90  0.443   7.22  20.32   9.00  1.49         809.8  SILTY SAND     MEDIUM RIGIDITY
 3.40  4.20  15.80   431.2  0.15  3.24   2.00  0.463   7.76  23.33  10.80  1.57         983.3  SILTY SAND     RIGID
 3.60  5.20  13.50   501.2  0.17  3.09   2.00  0.484   9.08  31.45  15.19  1.73        1222.5  SILTY SAND     RIGID
 3.80  5.50  19.80   540.0  0.19  3.15   2.00  0.503   9.17  32.89  16.14  1.74        1309.6  SILTY SAND     RIGID
 4.00  3.10   6.80   123.2  0.21  1.27   1.80  0.519   5.25   4.27   2.53  1.20         239.5  SANDY SILT     MEDIUM DENSITY
 4.20  3.00   6.40   116.5  0.23  1.20   1.80  0.535   4.90   4.65   2.17  1.14         209.4  SANDY SILT     MEDIUM DENSITY
 4.40  3.00   9.10   221.4  0.25  2.43   1.90  0.553   4.46   5.02   4.47  1.07         391.5  SILTY SAND     MEDIUM RIGIDITY
 4.60  3.00   6.10   104.9  0.27  1.09   1.70  0.567   4.53   3.64   2.06  1.09         180.8  SILT           LOW DENSITY
 4.80  3.10   6.00    97.1  0.29  0.98   1.70  0.581   4.62   3.64   2.15  1.10  0.36   167.8  SILT           LOW DENSITY
 5.00  3.50   6.40    97.1  0.31  0.86   1.80  0.597   5.13   4.35   2.60  1.18  0.43   177.5  SILT           MEDIUM DENSITY
 5.20  4.60   8.20   124.3  0.33  0.82   1.80  0.613   6.70   6.60   4.05  1.42  0.61   260.6  SILT           MEDIUM DENSITY
 5.40  4.40   7.90   120.4  0.35  0.84   1.80  0.629   6.19   5.83   3.67  1.35  0.57   242.8  SILT           MEDIUM DENSITY
 5.60  4.20   7.40   108.8  0.37  0.80   1.80  0.645   5.72   5.15   3.32  1.28  0.53   210.5  CLAYEY SILT    MEDIUM DENSITY
 5.80  4.80  10.90   221.4  0.39  1.45   1.90  0.661   6.24   7.52   5.23  1.35         452.4  SANDY SILT     MEDIUM DENSITY
 6.00  4.10   7.60   120.4  0.41  0.92   1.80  0.677   5.22   4.47   3.03  1.20  0.49   222.5  SILT           MEDIUM DENSITY


 6.20  4.90   9.90   143.7  0.43  0.91   1.80  0.693   6.18   5.82   4.03  1.35  0.63   289.9  SILT           MEDIUM DENSITY
 6.40  4.20   8.10   136.0  0.45  1.03   1.80  0.709   5.04   4.23   3.00  1.17         247.1  SILT           MEDIUM DENSITY
 6.60  4.30   6.80    81.6  0.47  0.59   1.80  0.725   5.14   4.36   3.16  1.18  0.52   148.3  SILTY CLAY     MEDIUM CONSISTENCY
 6.80  4.60   8.60   139.9  0.49  0.96   1.80  0.741   5.30   4.53   3.39  1.21  0.55   260.8  SILT           MEDIUM DENSITY
 7.00  4.00   7.70   128.2  0.51  1.04   1.80  0.757   4.39   3.41   2.58  1.06         215.4  SILT           MEDIUM DENSITY
 7.20  3.60   6.40    93.2  0.53  0.65   1.70  0.771   3.83   2.75   2.12  0.95  0.38   142.8  SILT           LOW DENSITY
 7.40  2.60   5.20    85.5  0.55  1.19   1.70  0.785   2.47   1.39   1.09  0.66          95.8  SILT           LOW DENSITY
 7.60  3.40   6.40   101.0  0.57  1.01   1.70  0.799   3.38   2.27   1.81  0.86         143.1  SILT           LOW DENSITY
 7.80  2.90   5.80    97.1  0.59  1.20   1.70  0.813   2.69   1.59   1.29  0.72         117.0  SANDY SILT     LOW DENSITY
 8.00  3.20   5.70    81.6  0.61  0.89   1.70  0.827   3.00   1.89   1.56  0.79  0.30   105.3  SILT           LOW DENSITY
 8.20  3.50   7.20   128.2  0.63  1.28   1.80  0.843   3.21   2.24   1.89  0.83         177.6  SANDY SILT     MEDIUM DENSITY
 8.40  3.60  10.10   237.0  0.65  2.42   1.90  0.861   3.07   3.97   3.42  0.80         339.0  SILTY SAND     MEDIUM RIGIDITY
 8.60  3.70  11.20   275.8  0.67  2.79   1.90  0.879   3.04   3.90   3.43  0.79         400.0  SILTY SAND     MEDIUM RIGIDITY
 8.80  3.50  11.10   279.7  0.69  3.09   1.90  0.897   2.73   3.17   2.84  0.73         384.0  SILTY SAND     MEDIUM RIGIDITY
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FILE NAME:     PIEZOBLADE RESEARCH
FILE NUMBER:   6

RECORD OF DILATOMETER TEST NO. D-6
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GEO, MARCH 1980

     LOCATION: LAKE ALICE (NORTH)
     PERFORMED:  DATE:  3 AUGUST 1981
                 BY  :  ALIREZA BOGHRAT


     CALIBRATION INFORMATION:
     DA=  0.22 BARS     DB=  0.21 BARS     ZM=  0.21 BARS     ZW=  0.55 METERS


  Z     A     B     ED     UO    ID   GAMMA   SV     KD    OCR   PC    KO   CU     M     SOIL TYPE     DESCRIPTION
***** ***** ***** ****** ***** ***** ***** ****** ****** ***** ***** **** ***** ****** ************ ******************
 0.20  1.65 11.50  366.0  0.00  8.32  1.80  0.035  33.97 ***** 13.69 3.73        1391.7     SAND      LOW RIGIDITY
 0.40  1.10  4.90  130.9  0.00  3.76  1.80  0.071  13.26 64.85  4.60 2.19         354.4     SAND      LOW RIGIDITY
 0.60  0.90  4.70  130.9  0.00  4.80  1.70  0.100   7.37 21.09  2.11 1.51         292.5     SAND          LOOSE
 0.80  1.10  5.70  162.0  0.02  5.00  1.80  0.116   7.56 22.15  2.57 1.54         365.6     SAND      LOW RIGIDITY
 1.00  1.50  7.35  210.6  0.04  4.77  1.80  0.132   9.05 31.23  4.12 1.73         508.1     SAND      LOW RIGIDITY
 1.20  1.65 10.90  342.7  0.06  8.03  1.80  0.148   7.80 23.52  3.48 1.57         782.6     SAND      LOW RIGIDITY



 1.40  5.70 18.15  467.0  0.08  2.51  2.00  0.168  29.90 ***** 51.50 3.48        1582.8  SILTY SAND       RIGID
 1.60  7.10 24.45  657.3  0.11  2.88  2.00  0.188  32.76 ***** 68.59 3.66        2282.2  SILTY SAND       RIGID
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FILE NAME:      PIEZOBLADE RESEARCH
FILE NUMBER:    7

RECORD OF DILATOMETER TEST NO. D-7
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GEO, MARCH 1980

     LOCATION: LAKE WALBURG (NORTH WEST)
     PERFORMED:  DATE:  14 AUGUST 1981
                 BY  :   ALIREZA BOGHRAT


     CALIBRATION INFORMATION:
     DA=  0.13 BARS     DB=  0.56 BARS     ZM=  0.21 BARS     ZW=  0.50 METERS


  Z      A      B      ED     UO     ID   GAMMA    SU      KD    CCR    PC    KO    CU      M      SOIL TYPE      DESCRIPTION
##### ##### ##### ###### ##### ##### ##### ###### ###### ##### ##### #### ##### ###### ############ #################
 0.20  1.25  9.10  278.2  0.00  9.26  1.80  0.035  23.20 *****  6.61 3.02        898.7     SAND      LOW RIGIDITY
 0.40  0.90  3.10   58.7  0.00  2.13  1.70  0.069  10.79 43.74  3.02 1.93        144.8  SILTY SAND       LOOSE
 0.60  0.50  1.20    0.4  0.01  0.03  1.50  0.089   4.60  3.67  0.33 1.09  0.06    0.7     MUD
 0.80  0.85  5.50  153.8  0.03  7.67  1.70  0.103   5.26 11.10  1.14 1.20        299.8     SAND            LOOSE
 1.00  1.90  4.85   87.8  0.05  1.43  1.70  0.117  14.16 30.23  3.54 2.27        238.3  SANDY SILT   LOW DENSITY
 1.20  1.45  3.90   68.4  0.07  1.52  1.60  0.129   9.40 17.34  2.24 1.77        166.8  SANDY SILT   COMPRESSIBLE



 1.40  2.30  5.70  105.3  0.09  1.43  1.70  0.143  13.95 29.22  4.18 2.25        284.3  SANDY SILT   LOW DENSITY
 1.60  2.10  5.10   89.7  0.11  1.35  1.70  0.157  11.43 18.85  2.96 2.00        226.2  SANDY SILT   LOW DENSITY
 1.80  2.30  6.80  148.0  0.13  2.11  1.80  0.173  10.98 45.22  7.82 1.95        367.7  SILTY SAND   LOW RIGIDITY
 2.00  4.40  9.30  163.6  0.15  1.12  1.80  0.189  20.95 39.03  7.38 2.85        501.8     SILT     MEDIUM DENSITY
 2.20  8.90 19.80  396.7  0.17  1.32  1.95  0.208  39.13 ***** 27.10 4.03       1440.8  SANDY SILT       DENSE
 2.40  3.40  8.30  163.6  0.19  1.51  1.80  0.224  13.03 29.79  6.67 2.16        431.7  SANDY SILT   MEDIUM DENSITY
 2.60  3.70  6.50   92.9  0.21  0.67  1.50  0.240  13.77 20.28  4.87 2.23  0.59  220.4  CLAYEY SILT  MEDIUM DENSITY
 2.80  3.90  7.10   97.5  0.23  0.76  1.50  0.256  13.53 19.74  5.05 2.21  0.61 ·260.7  CLAYEY SILT  MEDIUM DENSITY
 3.00  5.30 12.40  249.0  0.25  1.45  1.95  0.275  16.91 41.60 11.44 2.52        715.8  SANDY SILT       DENSE
 3.20  9.20 19.30  365.6  0.27  1.18  1.95  0.294  28.50 63.65 18.55 3.39       1223.3     SILT          DENSE
 3.40  8.70 19.40  383.9  0.29  1.34  1.95  0.313  25.01 66.27 20.74 3.15       1255.4  SANDY SILT       DENSE
 3.60 18.20 32.50  528.7  0.31  0.83  2.10  0.335  51.13 ***** 52.60 4.65  4.24 2048.4     SILT        VERY DENSE
 3.80 10.10 24.40  528.7  0.33  1.59  2.10  0.357  25.24 ***** 37.18 3.17       1711.1  SANDY SILT     VERY DENSE
 4.00 13.10 29.20  598.7  0.35  1.36  2.10  0.379  31.40 99.60 37.52 3.58       2055.6  SANDY SILT     VERY DENSE
 4.20 18.50 34.30  587.0  0.37  0.92  2.10  0.401  43.13 ***** 48.28 4.25  4.10 2183.9     SILT        VERY DENSE
 4.40 13.50 23.40  357.8  0.39  0.77  2.10  0.423  29.72 67.33 28.48 3.47  2.71 1210.7  CLAYEY SILT    VERY DENSE
 4.60 10.80 17.40  229.6  0.41  0.62  1.95  0.442  22.66 44.11 19.50 2.98  2.02  720.7  CLAYEY SILT      DENSE
 4.80 10.10 17.20  249.0  0.43  0.73  1.95  0.461  20.11 36.61 16.88 2.79  1.82  754.8  CLAYEY SILT      DENSE
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FILE NAME:     PIEZOBLADE RESEARCH
FILE NUMBER:   8

RECORD OF DILATOMETER TEST NO. D-8
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GED, MARCH 1980

     LOCATION: LAKE WALBURG (NORTH WEST)
     PERFORMED:  DATE:  15 AUGUST 1981
                 BY  :  ALIREZA BOGHRAT


     CALIBRATION INFORMATION:
     DA=  0.13 BARS     DB=  0.45 BARS     ZM=  0.21 BARS     ZW=  0.85 METERS

  Z     A     B      ED     UO    ID   GAMMA   SU      KD    CCR    PC    KO    CU      M      SOIL TYPE        DESCRIPTION
##### ##### ##### ###### ##### ##### ##### ###### ###### ##### ##### #### ##### ###### ############ ##################
 0.20  3.10 12.50  342.7  0.00  3.57  1.90  0.035  73.69 ***** 60.06 5.64        1451.1     SAND      MEDIUM RIGIDITY
 0.40  3.20  9.90  237.8  0.00  2.28  1.90  0.073  38.55 ***** 36.34 4.00         660.4  SILTY SAND    MEDIUM RIGIDITY
 0.60  1.30  6.40  175.6  0.00  4.77  1.80  0.109   9.12 31.72  3.46 1.74         424.9     SAND        LOW RIGIDITY
 0.80  1.65  5.55  129.0  0.00  2.48  1.80  0.145   9.68 35.57  5.16 1.80         318.7  SILTY SAND      LOW RIGIDITY
 1.00  2.45  8.30  204.7  0.01  2.65  1.90  0.168  12.45 57.49  9.66 2.10         531.9  SILTY SAND    MEDIUM RIGIDITY
 1.20  1.75 11.40  352.4  0.03  8.06  1.80  0.184   6.42 16.23  2.99 1.38         745.3     SAND        LOW RIGIDITY


 1.40  1.30  9.80  307.7  0.05 10.81  1.80  0.200   3.85  6.09  1.22 0.96         513.8     SAND        LOW RIGIDITY
 1.60  2.40 17.10  548.6  0.07  9.63  1.90  0.218   7.06 19.45  4.24 1.47        1205.5     SAND      MEDIUM RIGIDITY
 1.80  3.65  9.15  191.1  0.09  1.60  1.60  0.234  13.80 37.45  8.77 2.24         514.3  SANDY SILT    MEDIUM DENSITY
 2.00  7.30 17.65  379.6  0.11  1.55  1.95  0.253  26.15 ***** 26.26 3.23        1240.6  SANDY SILT        DENSE
 2.20  3.10  5.30   62.9  0.13  0.61  1.70  0.267  10.50 13.29  3.55 1.90  0.47   153.8  CLAYEY SILT     LOW DENSITY
 2.40  4.15  6.50   68.8  0.15  0.49  1.80  0.283  13.52 19.71  5.58 2.21  0.68   193.2  SILTY CLAY  MEDIUM CONSISTENCY
 2.60  4.10  7.10   94.0  0.17  0.68  1.80  0.299  12.45 17.34  5.19 2.10  0.65   244.3  CLAYEY SILT   MEDIUM DENSITY
 2.80  4.15  6.80   80.4  0.19  0.58  1.80  0.315  11.97 16.31  5.14 2.05  0.65   215.9  SILTY CLAY  MEDIUM CONSISTENCY
 3.00  4.30  6.40   59.1  0.21  0.41  1.80  0.331  11.87 16.09  5.33 2.04  0.67   158.0  SILTY CLAY  MEDIUM CONSISTENCY
 3.20  5.10  6.45   29.9  0.23  0.17  1.70  0.345  13.76 20.26  6.99 2.23  0.65    84.6     CLAY      LOW CONSISTENCY
 3.40  6.15  9.10   92.1  0.25  0.44  1.80  0.361  15.78 23.09  9.06 2.42  1.05   273.2  SILTY CLAY  MEDIUM CONSISTENCY
 3.60  7.75 14.10  224.2  0.27  0.85  1.95  0.380  18.70 32.70 12.43 2.67  1.37   664.3     SILT           DENSE
 3.80  8.30 12.30  132.9  0.29  0.46  1.90  0.398  19.48 34.65 13.87 2.74  1.51   423.0  SILTY CLAY   HIGH CONSISTENCY
 4.00  8.50 13.40  167.8  0.31  0.57  1.90  0.416  18.96 33.42 13.90 2.70  1.52   529.7  SILTY CLAY   HIGH CONSISTENCY
 4.20 11.75 22.35  389.3  0.33  0.97  2.10  0.438  24.74 50.58 22.15 3.13  2.23  1252.7     SILT         VERY DENSE
 4.40 11.50 23.80  455.3  0.35  1.17  2.10  0.460  22.78 44.48 20.46 2.99        1431.4     SILT         VERY DENSE
 4.60 16.80 33.10  610.7  0.37  1.06  2.10  0.482  32.28 76.62 36.93 3.63        2112.2     SILT         VERY DENSE
 4.80 19.45 37.80  690.4  0.39  1.03  2.10  0.504  35.89 90.38 45.55 3.85        2453.7     SILT         VERY DENSE
 5.00 11.20 27.20  599.1  0.41  1.63  2.10  0.521  19.07 69.05 35.98 2.70        1787.0  SANDY SILT      VERY DENSE
 5.20 10.60 19 70  331.0  0.43  0.93  1.95  0.540  17.89 30.50 16.47 2.61  1.84   968.3     SILT           DENSE
 5.40 10.50 19.40  323.2  0.45  0.91  1.95  0.559  17.08 28.39 15.87 2.54  1.80   932.1     SILT           DENSE
 5.60 12.80 23.40  389.3  0.47  0.90  2.10  0.581  20.21 36 91 21.45 2.80  2.30  1181.7     SILT         VERY DENSE
 5.80 10.55 21.45  400.9  0.49  1.15  1.95  0.600  15.77 25.05 15.03 2.42        1127.1     SILT           DENSE
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FILE NAME:     PIEZOBLADE RESEARCH
FILE NUMBER:   9

RECORD OF DILATOMETER TEST NO. D-9
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GEO, MARCH 1980

     LOCATION: LAKE VAUBURG (NORTH WEST)
     PERFORMED:  DATE:  15 AUGUST 1981
                 BY  :  ALIREZA SOSHRAT


     CALIBRATION INFORMATION:
     DA=  0.20 BARS     DB=  0.45 BARS     ZM=  0.21 BARS     ZW=  0.90 METERS


  Z     A     B     ED    UO    ID   GAMMA   SV      KD    OCR    PC   KO   CU      M     SOIL TYPE        DESCRIPTION
##### ##### ##### ###### ##### ##### ##### ###### ###### ##### ##### #### ##### ###### ############## ####################
 0.20  1.75  8.75 [illegible] [illegible] [illegible] 1.80 [illegible] [illegible] ----- [illegible] [illegible]  [illegible]     SAND         LOW RIGIDITY
 0.40  1.80 10.70  320.5  0.00  6.29  1.80  0.071  19.40 ***** 3.52 2.73           985.6     SAND         LOW RIGIDITY
 0.60  2.75 14.10  415.7  0.00  5.10  1.90  0.109  20.23 ***** 15.84 2.89         1293.6     SAND       MEDIUM RIGIDITY
 0.80  2.30 15.20  456.3  0.00  5.60  1.90  0.147  14.98 51.86 12.04 2.35         1301.5     SAND       MEDIUM RIGIDITY
 1.00  2.20  4.50   75.3  0.01  0.98  1.70  0.171  12.18 15.75  2.36 2.98  0.35   145.3     SILT         LOW DENSITY
 1.20  2.75  4.90   58.3  0.03  0.60  1.70  0.185  14.24 21.33  3.96 2.28  0.47   166.8  SILTY CLAY   LOW CONSISTENCY



 1.40  2.?5  3.?? [illegible] 0.05  0.36  1.70  0.199  11.61 15.?? 3.18 2.04  0.40    83.0  SILTY CLAY   LOW CONSISTENCY
 1.60  2.50  4.?? [illegible] 0.07  0.42  1.70  0.213  11.1? ??.57 3.10 1.47  0.40    96.3  SILTY CLAY   LOW CONSISTENCY
 1.80  2.?? [illegible] 38.? 0.09  0.39  1.70  0.227  11.99 16.14  3.46 2.05  0.45   104.0  SILTY CLAY   LOW CONSISTENCY
 2.00  2.40  4.50   36.9  0.11  0.37  1.70  0.241  11.34 14.?8  3.61 1.99  0.46    97.0  SILTY CLAY   LOW CONSISTENCY
 2.20  3.65  5.40   42.7  0.13  0.33  1.70  0.255  13.55 19.78  5.04 2.21  0.61   120.1  SILTY CLAY   LOW CONSISTENCY
 2.40  4.35  8.50  151.5  0.15  1.03  1.80  0.271  14.74 22.56  6.11 2.33         416.8     SILT       MEDIUM DENSITY
 2.60  4.30  8.10  122.4  0.17  0.83  1.80  0.287  13.81 20.37  5.85 2.24  0.71   329.4     SILT       MEDIUM DENSITY
 2.80  4.60  8.80  137.9  0.19  0.86  1.80  0.303  13.94 20.66  6.26 2.25  0.75   372.3     SILT       MEDIUM DENSITY
 3.00  7.10 12.25  174.3  0.21  0.71  1.93  0.322  20.67 33.32 12.31 2.53  1.31   534.2  CLAYEY SILT       DENSE
 3.20  8.90 15.20  219.5  0.23  0.71  1.95  0.341  24.57 50.04 17.07 3.12  1.73   705.0  CLAYEY SILT       DENSE
 3.40 15.80 25.30  343.8  0.25  0.62  2.10  0.363  41.59 ***** 41.30 4.17  3.55  1267.6  CLAYEY SILT    VERY DENSE
 3.60 17.65 26.85  332.2  0.27  0.53  2.05  0.384  44.12 ***** 47.90 4.30  4.09  1335.8  SILTY CLAY        HARD
 3.80 [illegible] 0.29 [illegible] 0.40? [illegible]  SILTY CLAY        HARD
 4.00 [illegible] 0.31 [illegible] 0.424 [illegible]  SILTY CLAY        HARD
 4.20 [illegible] 13.?5 132.? 0.33 0.?1 1.?0 0.444 21.?? [illegible] 1.?? ?03.?  SILTY CLAY  HIGH CONSISTENCY
 4.40  9.?0 14.?5  170.4  0.3?  0.50  1.?0  0.462  ??.17 [illegible] 1.53  550.3  SILTY CLAY  HIGH CONSISTENCY
 4.60 [illegible] 25.39 [illegible] 0.37 0.55 2.?5 0.483 32.62 77.?8 37.61 3.65 3.43 1196.9 SILTY CLAY   HARD
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FILE NAME:     PIEZOBLADE RESURCH
FILE NUMBER:   10

RECORD OF DILATOMETER TEST NO. D-10
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GED, MARCH 1980

     LOCATION: LAKE ALICE(NORTH)
     PERFORMED:  DATE:  23 OCT. 1981
                 BY  :  ALIREZA BOGHRAT


     CALIBRATION INFORMATION:
     DA=  0.11 BARS     DB=  0.48 BARS     ZM=  0.10 BARS     ZW=  0.80 METERS


  Z     A     B      ED    UO    ID   GAMMA   SU      KD   CCR    PC    KO    CU      M     SOIL TYPE        DESCRIPTION
##### ##### ##### ###### ##### ##### ##### ###### ###### ##### ##### #### ##### ###### ############ ####################
 0.20  1.50  8.35  243.2  0.00  5.49  1.80  0.035  34.20 ***** 13.86 3.75         867.8     SAND          LOW RIGIDITY
 0.40  1.95  7.90  208.2  0.00  3.33  1.80  0.071  23.83 ***** 14.11 3.07         677.7     SAND          LOW RIGIDITY
 0.60  1.65  6.40  161.6  0.00  3.01  1.80  0.107  13.57 67.77  7.25 2.22         446.9  SILTY SAND       LOW RIGIDITY
 0.80  2.10  7.15  173.3  0.00  2.48  1.80  0.143  13.20 64.24  9.19 2.18         459.3  SILTY SAND       LOW RIGIDITY
 1.00  1.95  7.30  184.9  0.02  2.94  1.90  0.159  10.70 43.08  6.85 1.92         455.2  SILTY SAND       LOW RIGIDITY
 1.20  7.15 18.85  431.6  0.04  1.78  1.95  0.178  36.88 ***** 52.14 3.90        1544.7  SANDY SILT          DENSE



 1.40  8.10 18.15  367.5  0.06  1.31  1.95  0.197  38.46 ***** 24.66 3.99        1329.3  SANDY SILT          DENSE
 1.60  3.80 10.55  239.3  0.08  1.89  1.90  0.215  15.92 76.73 16.51 2.43         674.8  SILTY SAND     MEDIUM RIGIDITY
 1.80  5.30 10.15  165.5  0.10  0.90  1.80  0.231  21.63 41.03  9.48 2.91  1.00   512.6     SILT        MEDIUM DENSITY
 2.00  5.50 10.45  169.4  0.12  0.89  1.80  0.247  20.54 37.00  9.63 2.85  1.02   519.6     SILT        MEDIUM DENSITY
 2.20  4.70  9.95  181.0  0.14  1.13  1.80  0.263  16.49 26.97  7.07 2.49         516.3     SILT        MEDIUM DENSITY
 2.40  4.25 10.50  219.9  0.16  1.56  1.80  0.279  13.68 34.55  9.64 2.23         590.0  SANDY SILT     MEDIUM DENSITY
 2.60  5.95 13.75  280.1  0.18  1.40  1.95  0.298  18.19 43.21 12.88 2.63         823.6  SANDY SILT          DENSE
 2.80  3.80  6.95   99.5  0.20  0.77  1.60  0.314  11.09 14.47  4.54 1.96  0.59   248.0  CLAYEY SILT    MEDIUM DENSITY
 3.00  4.30  6.60   66.4  0.22  0.45  1.80  0.330  12.13 16.65  5.50 2.07  0.69   179.3  SILTY CLAY   MEDIUM CONSISTENCY
 3.20  4.90  8.65  122.8  0.24  0.74  1.80  0.346  13.04 18.63  6.45 2.16  0.79   324.1  CLAYEY SILT    MEDIUM DENSITY
 3.40  5.90 10.30  148.0  0.26  0.73  1.80  0.362  15.08 23.33  8.46 2.36  1.00   410.2  CLAYEY SILT    MEDIUM DENSITY
 3.60  5.20 10.15  169.4  0.28  0.97  1.80  0.378  12.47 17.37  6.56 2.11  0.82   440.2     SILT        MEDIUM DENSITY
 3.80  6.70 10.60  128.6  0.30  0.56  1.80  0.394  15.85 25.26  9.95 2.43  1.15   382.2  SILTY CLAY   MEDIUM CONSISTENCY
 4.00  8.10 13.65  192.7  0.32  0.69  1.95  0.413  18.26 31.51 13.01 2.64  1.44   567.3  CLAYEY SILT         DENSE
 4.20  7.95 12.25  144.1  0.34  0.52  1.90  0.431  17.25 28.92 12.42 2.55  1.40   440.9  SILTY CLAY    HIGH CONSISTENCY
 4.40  8.50 13.45  169.4  0.36  0.58  1.90  0.449  17.67 29.92 13.43 2.59  1.50   522.3  SILTY CLAY    HIGH CONSISTENCY
 4.60  8.95 15.50  231.5  0.38  0.76  1.95  0.468  17.70 30.00 14.04 2.59  1.57   673.1  CLAYEY SILT         DENSE
 4.80 10.30 16.30  210.2  0.40  0.59  1.90  0.486  19.83 35.64 17.42 2.77  1.88   673.0  SILTY CLAY    HIGH CONSISTENCY
 5.00  7.15 11.30  139.3  0.42  0.57  1.90  0.504  13.02 19.59  9.37 2.16  1.15   383.2  SILTY CLAY    HIGH CONSISTENCY
 5.20  8.60 13.15  153.8  0.44  0.52  1.90  0.522  15.27 23.84 12.44 2.38  1.46   451.4  SILTY CLAY    HIGH CONSISTENCY
 5.40  7.30 11.55  142.2  0.46  0.58  1.90  0.540  12.35 17.11  9.24 2.09  1.16   386.2  SILTY CLAY    HIGH CONSISTENCY
 5.60  6.50 13.60  252.9  0.48  1.20  1.95  0.559  10.20 12.71  7.10 1.86         611.6     SILT            DENSE
 5.80  7.30 15.95  313.1  0.50  1.32  1.95  0.578  11.08 17.16  9.92 1.96         780.6  SANDY SILT          DENSE
 6.00  8.25 16.95  315.1  0.52  1.16  1.95  0.597  12.29 16.98 10.14 2.09         614.8     SILT            DENSE



 6.20  7.80 17.85  367.5  0.54  1.46  1.95  0.616  11.03 20.82 12.82 1.95         914.7  SANDY SILT          DENSE
 6.40  7.65 17.50  359.8  0.56  1.46  1.95  0.635  10.45 19.11 12.13 1.89         877.8  SANDY SILT          DENSE
 6.60  6.10 14.10  287.9  0.58  1.51  1.95  0.654   7.89 12.61  8.24 1.58         653.8  SANDY SILT          DENSE
 6.80  7.70 17.65  363.6  0.60  1.48  1.95  0.673   9.87 17.72 11.92 1.82         904.5  SANDY SILT          DENSE
 7.00  7.50 22.15  546.2  0.62  2.39  2.00  0.693   8.93 30.46 21.11 1.71        1309.3  SILTY SAND          RIGID
 7.20  7.99 23.50  581.2  0.64  2.39  2.00  0.713   9.22 32.37 23.08 1.75        1410.0  SILTY SAND          RIGID
 7.40  8.20 20.95  472.4  0.66  1.84  2.00  0.733   9.47 27.19 19.93 1.78        1156.8  SILTY SAND          RIGID
 7.60  7.50 19.75  453.0  0.68  1.96  2.00  0.753   8.30 25.07 18.88 1.63        1053.2  SILTY SAND          RIGID
 7.80  8.15 19.10  402.5  0.70  1.57  1.95  0.772   8.99 17.04 13.16 1.72         965.1  SANDY SILT          DENSE
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FILE NAME:     PIEZOBLADE RESURCH
FILE NUMBER:   11

RECORD OF DILATOMETER TEST NO. D-11
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE. J. GED, MARCH 1980

     LOCATION: LAKE ALICE(NORTH)
     PERFORMED:  DATE:  24 OCT. 1981
                 BY  :  ALIREZA BOGHRAT


     CALIBRATION INFORMATION:
     DA=  0.10 BARS     DB=  0.43 BARS     ZM=  0.10 BARS     ZW=  0.82 METERS


  Z     A      B      ED    UO    ID  GAMMA    SU     KD    CCR    PC   KO    CU      M    SOIL TYPE      DESCRIPTION
##### ##### ##### ###### ##### ##### ##### ###### ###### ##### ##### #### ##### ###### ############ ###################
 0.20  1.50   8.20  239.7  0.00  5.44  1.80  0.035  34.04 ***** 13.74 3.74         854.3     SAND        LOW RIGIDITY
 0.40  1.95   7.50  195.0  0.00  3.10  1.80  0.071  23.93 ***** 14.22 3.08         635.4  SILTY SAND     LOW RIGIDITY
 0.60  1.55   6.60  175.6  0.00  3.58  1.80  0.107  12.37 56.82  6.08 2.10         471.5     SAND        LOW RIGIDITY
 0.80  1.70   6.90  181.4  0.00  3.34  1.90  0.143  10.26 37.69  5.68 1.87         457.5     SAND        LOW RIGIDITY
 1.00  2.30   8.25  210.6  0.02  2.83  1.90  0.163  12.34 56.50  9.21 2.09         545.3  SILTY SAND   MEDIUM RIGIDITY
 1.20  6.15  19.20  486.4  0.04  2.40  2.00  0.183  29.98 ***** 56.36 3.49        1649.8  SILTY SAND        RIGID


 1.40  6.20  24.35  684.5  0.06  3.52  2.00  0.203  25.92 ***** 47.34 3.22        2277.6     SAND           RIGID
 1.60  6.20  16.20  367.9  0.08  1.76  1.95  0.222  25.44 ***** 32.19 3.18        1193.4  SANDY SILT        DENSE
 1.80  7.10  14.45  265.0  0.10  1.08  1.95  0.241  27.64 60.14 14.49 3.33         879.2     SILT           DENSE
 2.00  6.25  15.15  325.2  0.12  1.54  1.95  0.260  21.98 75.41 19.61 2.93        1011.7  SANDY SILT        DENSE
 2.20  5.25  12.10  245.5  0.14  1.38  1.95  0.279  17.19 39.55 10.76 2.55         709.4  SANDY SILT        DENSE
 2.40  7.90  15.70  282.4  0.16  1.03  1.95  0.298  24.76 50.66 15.10 3.14         909.2     SILT           DENSE
 2.60  7.50  14.50  251.4  0.18  0.97  1.95  0.317  22.08 42.35 13.43 2.94  1.40   783.1     SILT           DENSE
 2.80  8.90  18.55  354.3  0.20  1.16  1.95  0.336  24.54 49.96 16.79 3.12        1137.7     SILT           DENSE
 3.00  8.40  14.70  224.2  0.22  0.77  1.95  0.355  22.24 42.83 15.21 2.95  1.59   699.8  CLAYEY SILT       DENSE
 3.20  8.70  17.75  331.0  0.24  1.11  1.95  0.374  21.49 40.60 15.19 2.89        1023.1     SILT           DENSE
 3.40  6.30  12.10  204.7  0.26  0.96  1.95  0.393  14.70 22.47  8.83 2.32  1.05   562.7     SILT           DENSE
 3.60  8.00  16.30  301.9  0.28  1.11  1.95  0.412  17.80 30.27 12.47 2.60         881.7     SILT           DENSE
 3.80  7.50  13.30  204.7  0.30  0.80  1.95  0.431  16.10 23.98 11.15 2.45  1.29   579.4  CLAYEY SILT       DENSE
 4.00  7.25  15.10  264.4  0.32  1.17  1.95  0.450  14.59 22.20  9.99 2.31         779.6     SILT           DENSE
 4.20  7.30  14.20  247.5  0.34  1.01  1.95  0.469  14.17 21.29  9.94 2.27         671.8     SILT           DENSE
 4.40  7.15  13.45  224.2  0.36  0.93  1.95  0.488  13.33 19.27  9.41 2.19  1.15   596.1     SILT           DENSE
 4.60  6.10  11.15  175.6  0.38  0.86  1.80  0.504  10.90 14.10  7.10 1.94  0.92   435.2     SILT       MEDIUM DENSITY
 4.80  6.35  18.85  465.0  0.40  2.35  2.00  0.524  10.22 39.41 20.65 1.86        1125.1  SILTY SAND        RIGID
 5.00  6.95  11.50  156.2  0.42  0.67  1.95  0.545  11.62 15.56  8.48 2.02  1.08   396.0  CLAYEY SILT       DENSE
 5.20  8.40  13.95  195.0  0.44  0.68  1.95  0.564  13.67 20.06 11.31 2.23  1.37   523.2  CLAYEY SILT       DENSE
 5.40  8.10  13.10  173.7  0.46  0.63  1.95  0.583  12.72 17.93 10.45 2.13  1.30   454.6  CLAYEY SILT       DENSE
 5.60  8.30  13.25  171.7  0.48  0.61  1.95  0.602  12.63 17.72 10.67 2.12  1.33   448.3  CLAYEY SILT       DENSE
 5.80  6.90  12.10  181.4  0.50  0.79  1.95  0.621   9.93 12.19  7.57 1.83  1.01   452.3  CLAYEY SILT       DENSE
 6.00  7.60  13.50  208.6  0.52  0.83  1.95  0.640  10.65 13.56  8.69 1.91  1.14   512.5     SILT           DENSE


 6.20  7.35  12.70  187.3  0.54  0.77  1.95  0.659   9.97 12.26  8.08 1.84  1.08   467.6  CLAYEY SILT       DENSE
 6.40  7.60  12.40  165.9  0.56  0.66  1.95  0.678  10.07 12.45  8.44 1.85  1.13   399.2  CLAYEY SILT       DENSE
 6.60  6.50  12.20  200.9  0.58  0.96  1.95  0.697   8.13  8.91  6.21 1.61  0.88   460.4     SILT           DENSE
 6.80  7.30  11.35  136.8  0.60  0.57  1.90  0.715   9.13 10.68  7.64 1.74  1.05   329.1  SILTY CLAY   HIGH CONSISTENCY
 7.00  8.25  12.90  160.1  0.62  0.58  1.90  0.733  10.13 12.57  9.21 1.85  1.23   402.3  SILTY CLAY   HIGH CONSISTENCY
 7.20  7.15  11.30  140.6  0.64  0.60  1.95  0.752   8.42  9.41  7.08 1.65  1.00   326.8  CLAYEY SILT       DENSE
 7.40  7.80  11.90  138.7  0.66  0.54  1.90  0.770   9.04 10.53  8.11 1.73  1.12   332.4  SILTY CLAY   HIGH CONSISTENCY
 7.60 10.20  17.90  278.6  0.68  0.82  1.95  0.784  11.61 15.55 12.27 2.02  1.56   706.2     SILT           DENSE
 7.80  9.55  17.10  272.7  0.70  0.87  1.95  0.808  10.52 13.33 10.77 1.90  1.42   667.1     SILT           DENSE
```

```
FILE NAME:     'PIEZOBLADE RESEARCH
FILE NUMBER:   1

RECORD OF DILATOMETER TEST NO. D-1
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE. J. GED, MARCH 1980

     LOCATION: LAKE ALICE (NORTH)
     PERFORMED:  DATE:  1 JULY 1981
                 BY  :  ALIREZA BOGHRAT


     CALIBRATION INFORMATION:
     DA=  0.20 BARS     DB=  0.20 BARS     ZM=  0.20 BARS     ZW=  0.40 METERS
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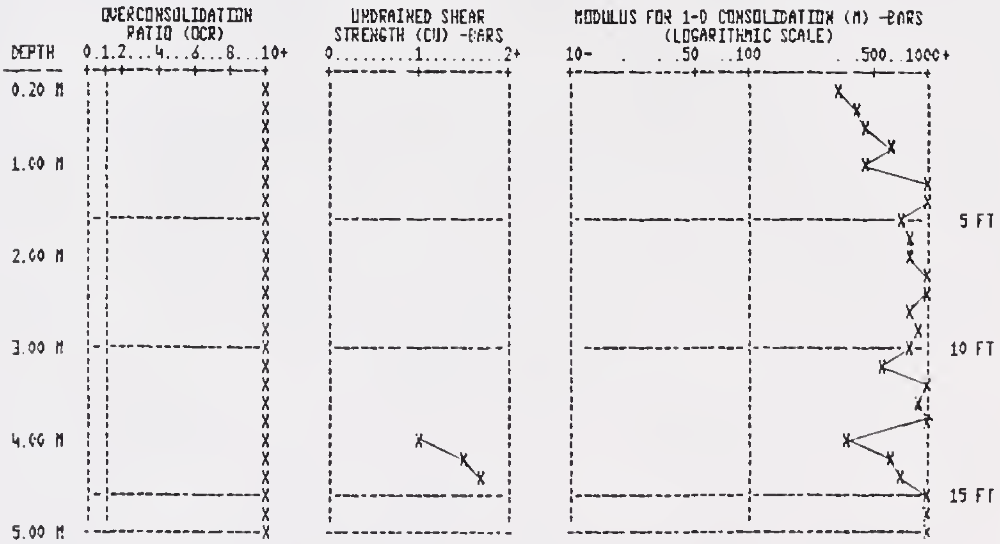

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FILE NAME:      PIEZOBLADE RESEARCH
FILE NUMBER:    2

RECORD OF DILATOMETER TEST NO. D-2
USING: DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GED, MARCH 1980

    LOCATION: LAKE ALICE (NORTH)
    PERFORMED:  DATE:  6 JULY 1981
                BY  :  ALIREZA BOGHRAT


    CALIBRATION INFORMATION:
    DA=  0.10 BARS     DB=  0.20 BARS     ZM=  0.20 BARS     ZW=  6.90 METERS
```

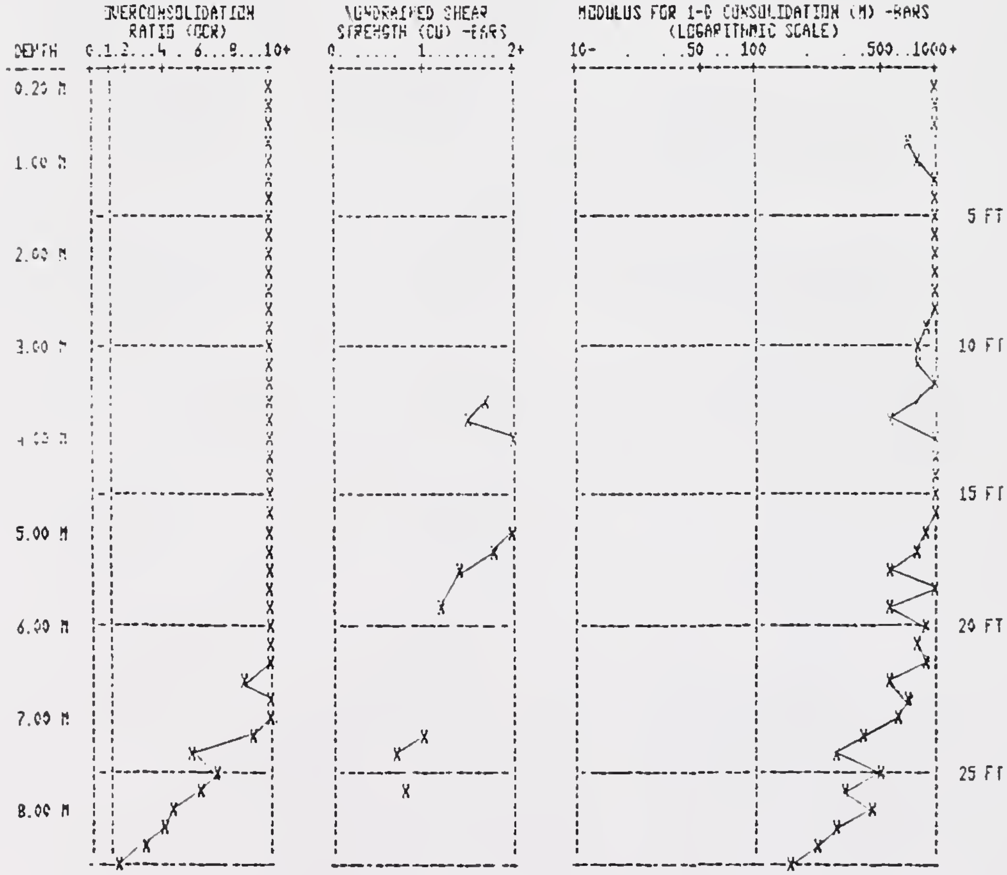

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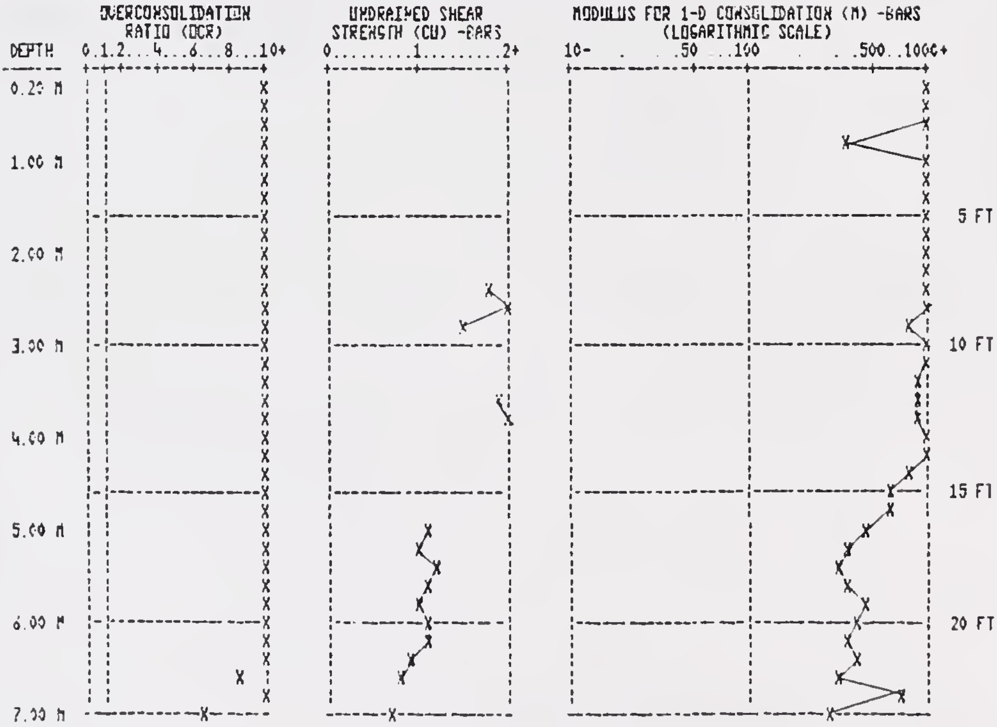
FILE NAME:     PIEZOBLADE RESEARCH
FILE NUMBER:   3

RECORD OF DILATOMETER TEST NO. D-3
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GED, MARCH 1980

    LOCATION: LAKE ALICE (EAST)
    PERFORMED:  DATE:  9 JULY 1981
                BY  :  ALIREZA BOGHRAT


    CALIBRATION INFORMATION:
    DA=  0.10 BARS     DB=  0.20 BARS     ZM=  0.20 BARS     ZW=  0.45 METERS
```



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FILE NAME:     PIEZOBLADE RESEARCH
FILE NUMBER:   4

RECORD OF DILATOMETER TEST NO. D-4
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE. J. GEO, MARCH 1980

    LOCATION: LAKE ALICE (WEST)
    PERFORMED:  DATE:  20 JULY 1981
                BY  :  ALIREZA BOGHRAT


    CALIBRATION INFORMATION:
    DA=  0.20 BARS     DB=  0.20 BARS     ZM=  0.20 BARS     ZW=  1.50 METERS
```

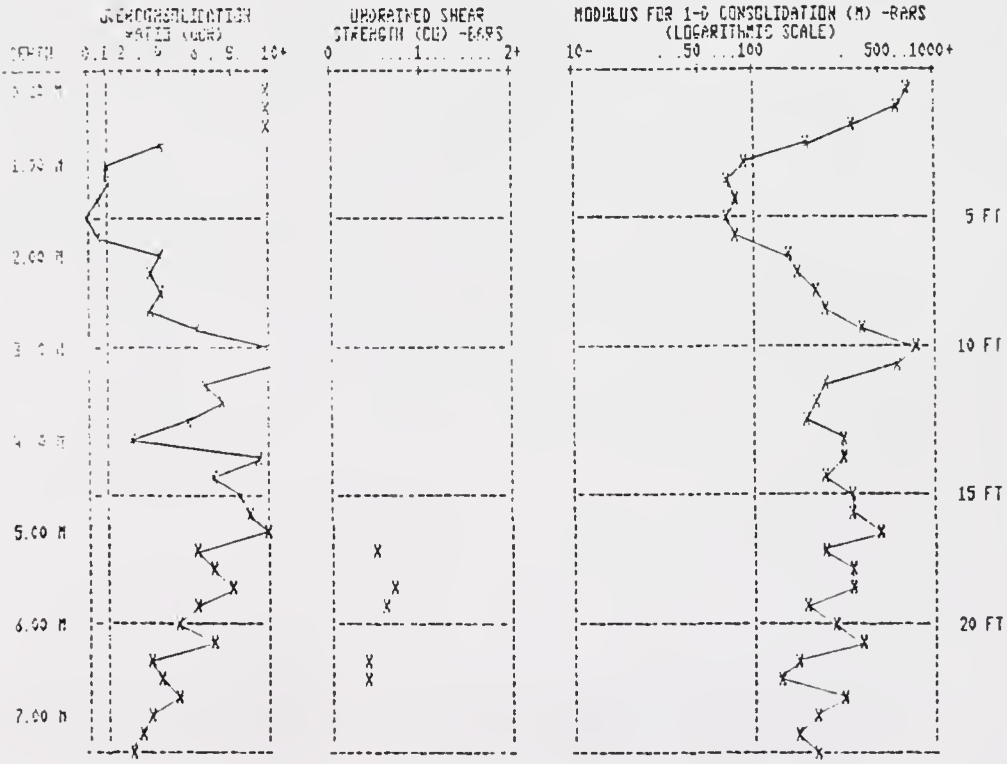

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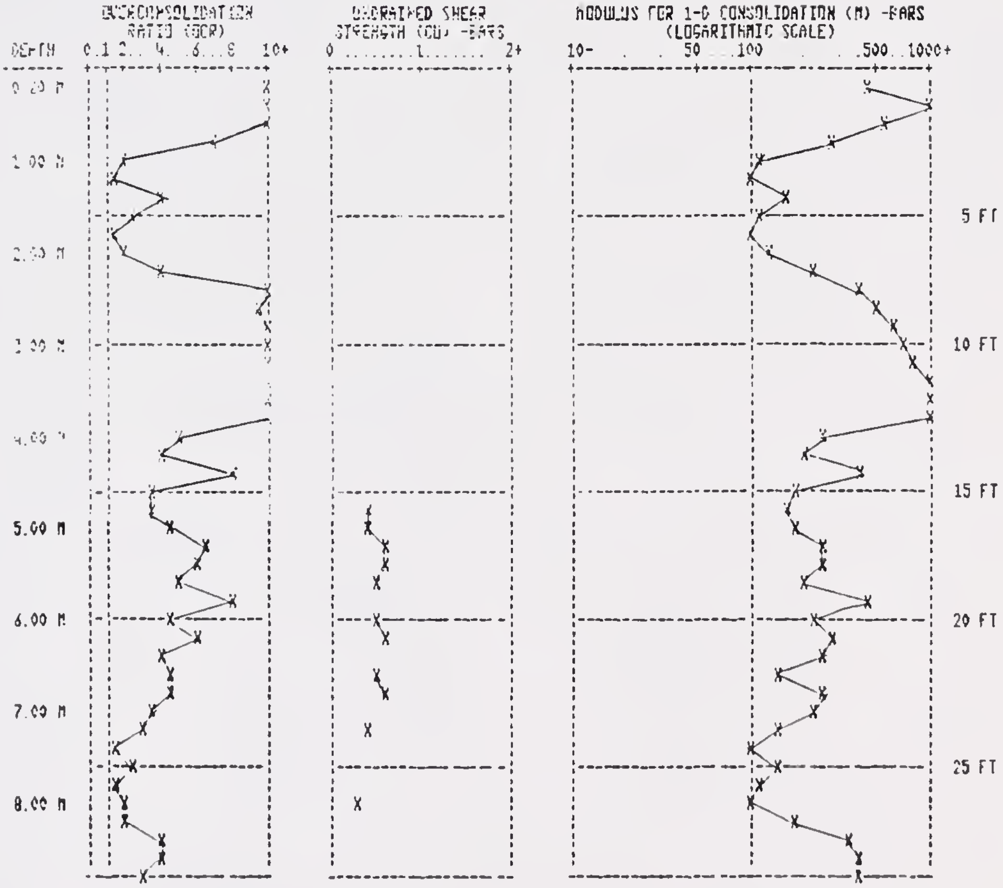
FILE NAME:      PIEZOBLADE RESEARCH
FILE NUMBER:    5

RECORD OF DILATOMETER TEST NO. D-5
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GEO, MARCH 1980

    LOCATION: LAKE ALICE (WEST)
    PERFORMED:  DATE:  20 JULY 1981
                BY  :  ALIREZA BOGHRAT


    CALIBRATION INFORMATION:
    DA=  0.20 BARS     DB=  0.20 BARS     ZM=  0.20 BARS     ZW=  1.90 METERS
```



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FILE NAME:     PIEZOBLADE RESEARCH
FILE NUMBER:   6

RECORD OF DILATOMETER TEST NO. D-6
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GED, MARCH 1980

     LOCATION: LAKE ALICE (NORTH)
     PERFORMED:  DATE:  3 AUGUST 1981
                 BY  :  ALIREZA BOGHRAT


     CALIBRATION INFORMATION:
     DA=  0.22 BARS     DB=  0.21 BARS     ZM=  0.21 BARS     ZW=  0.55 METERS
```

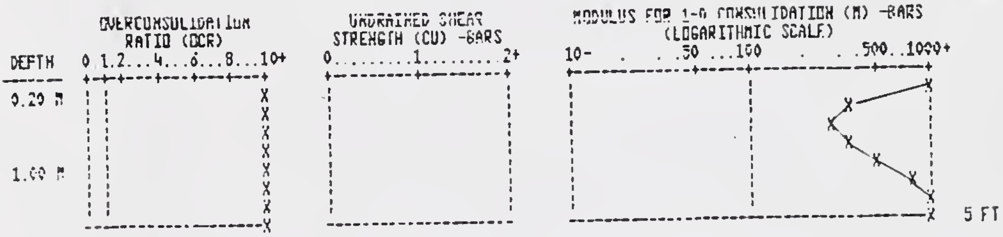

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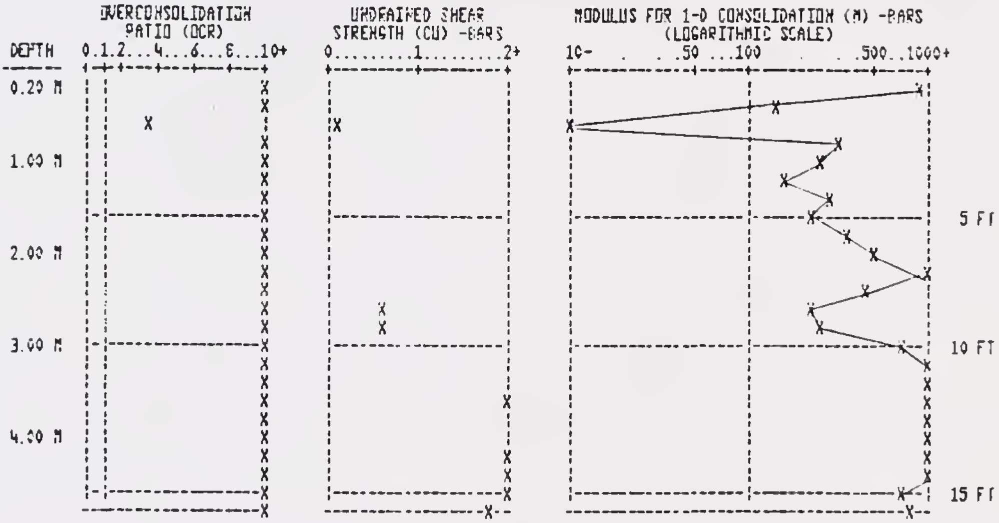
FILE NAME:     PIEZOBLADE RESEARCH
FILE NUMBER:   7

RECORD OF DILATOMETER TEST NO. D-7
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GEO, MARCH 1980

    LOCATION: LAKE WAUBURG (NORTH WEST)
    PERFORMED:  DATE:  14 AUGUST 1981
                BY  :   ALIREZA BOGHRAT


    CALIBRATION INFORMATION:
    DA=  0.13 BARS     DB=  0.56 BARS     ZM=  0.21 BARS     ZW=  0.50 METERS
```



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FILE NAME.      PIEZOBLADE RESEARCH
FILE NUMBER:    8

RECORD OF DILATOMETER TEST NO. D-8
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GEO, MARCH 1980

     LOCATION: LAKE WAUBURG (NORTH WEST)
     PERFORMED:  DATE:  15 AUGUST 1981
                 BY  :  ALIREZA BOGHRAT


     CALIBRATION INFORMATION:
     DA=  0.13 BARS     DB=  0.45 BARS     ZM=  0.21 BARS     ZW=  0.85 METERS
```
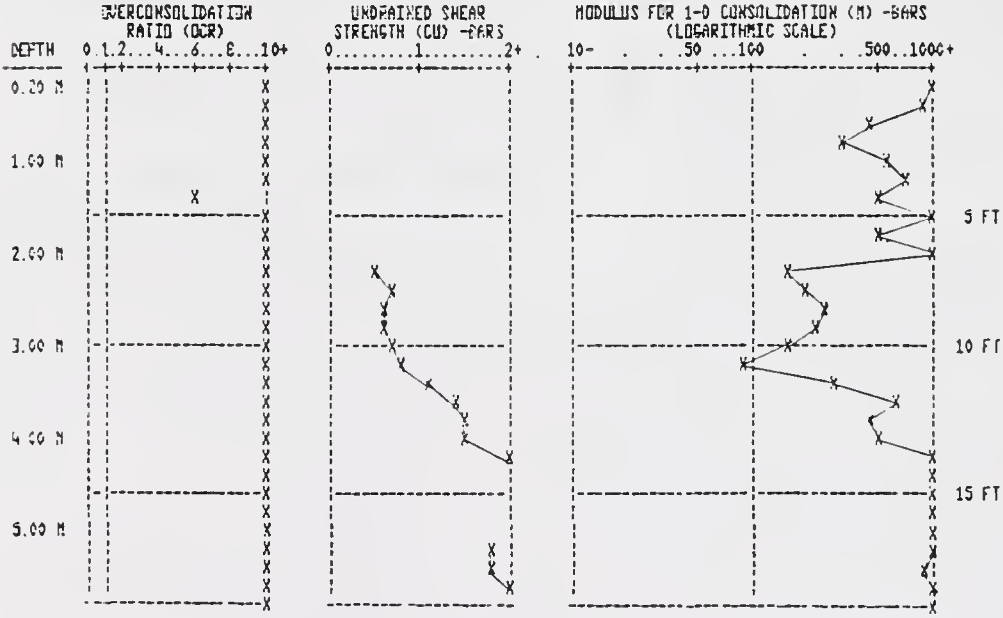
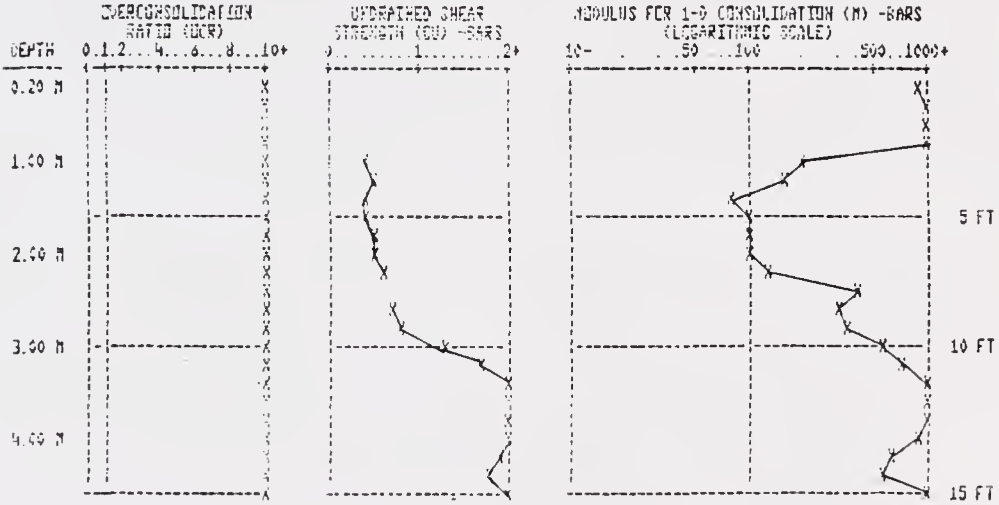
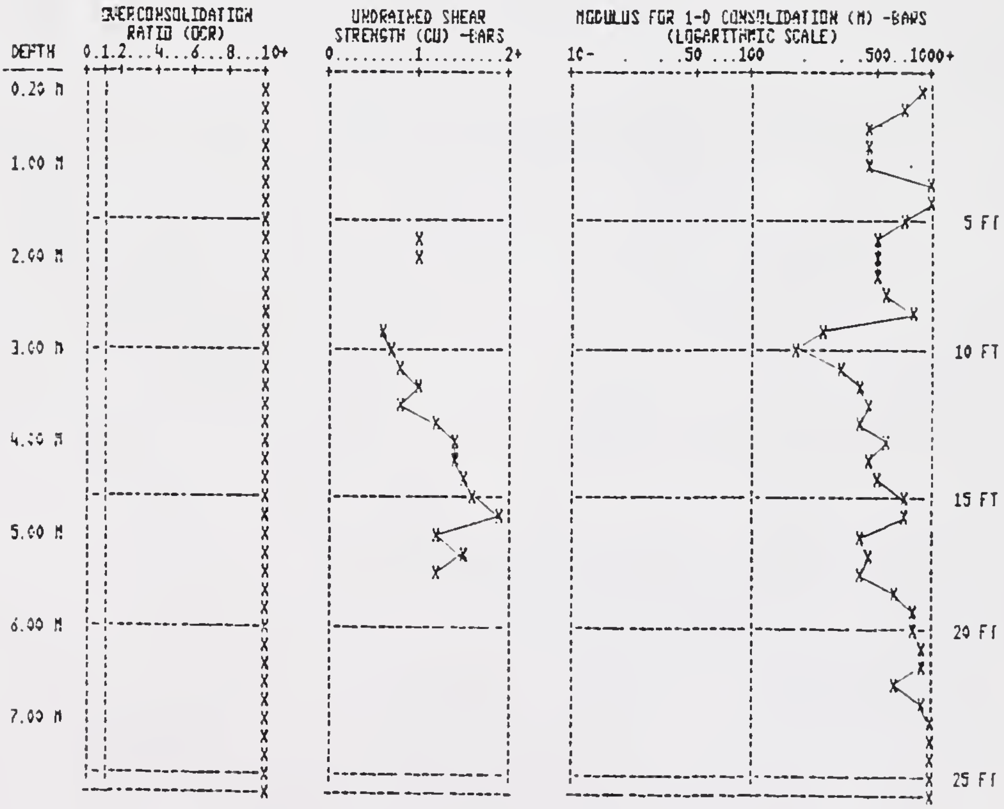



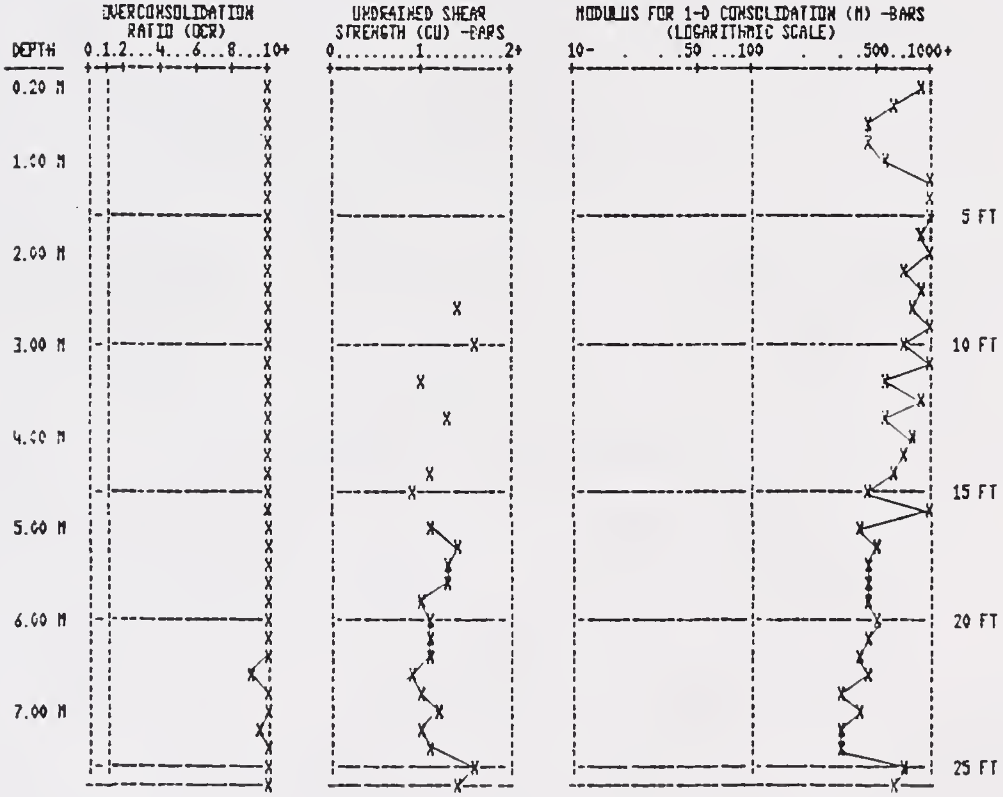
```
FILE NAME:     PIEZOBLADE RESEARCH
FILE NUMBER:   9

RECORD OF DILATOMETER TEST NO. D-9
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GEO, MARCH 1980

     LOCATION: LAKE WAUBURG (NORTH WEST)
     PERFORMED:  DATE:  15 AUGUST 1981
                 BY  :  ALIREZA BOGHRAT


     CALIBRATION INFORMATION:
     DA=  0.20 BARS     DB=  0.45 BARS     ZM=  0.21 BARS     ZV=  0.90 METERS
```



```
FILE NAME:     PIEZOBLADE RESURCH
FILE NUMBER:   10

RECORD OF DILATOMETER TEST NO. D-10
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GED, MARCH 1980

    LOCATION: LAKE ALICE(NORTH)
    PERFORMED:  DATE:  23 OCT. 1981
                BY  :  ALIREZA BOGHRAT


    CALIBRATION INFORMATION:
    DA=  0.11 BARS     DB=  0.48 BARS     ZM=  0.10 BARS     ZW=  0.80 METERS
```



```
FILE NAME:     PIEZOBLADE RESURCH
FILE NUMBER:   11

RECORD OF DILATOMETER TEST NO. D-11
USING DATA REDUCTION PROCEDURES IN  MARCHETTI, ASCE, J. GED, MARCH 1980

    LOCATION: LAKE ALICE(NORTH)
    PERFORMED:  DATE:  24 OCT. 1981
                BY  :  ALIREZA BOGHRAT

    CALIBRATION INFORMATION:
    DA=  0.10 BARS     DB=  0.43 BARS .   ZM=  0.10 BARS     ZW=  0.82 METERS
```

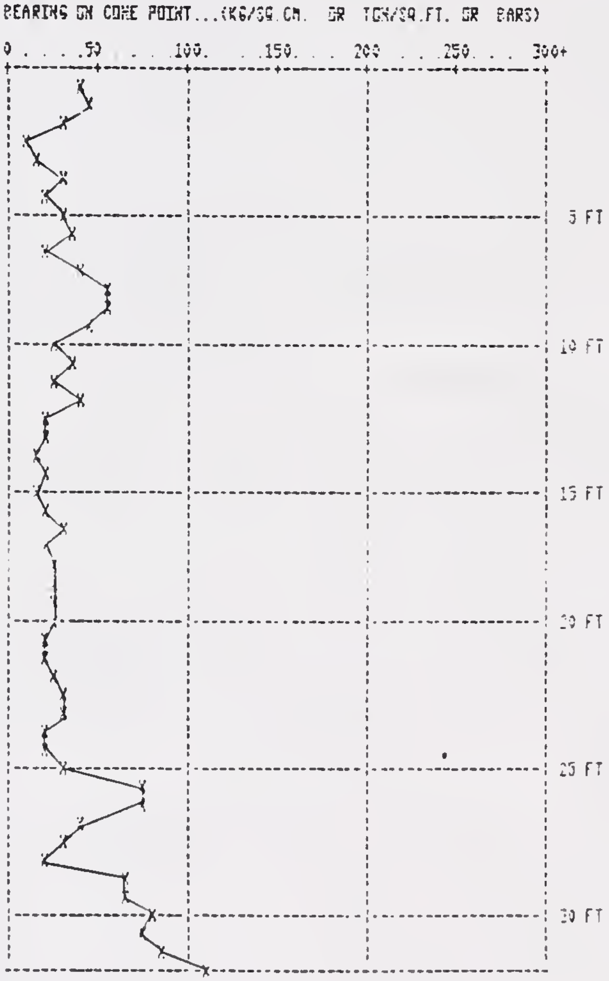


APPENDIX C

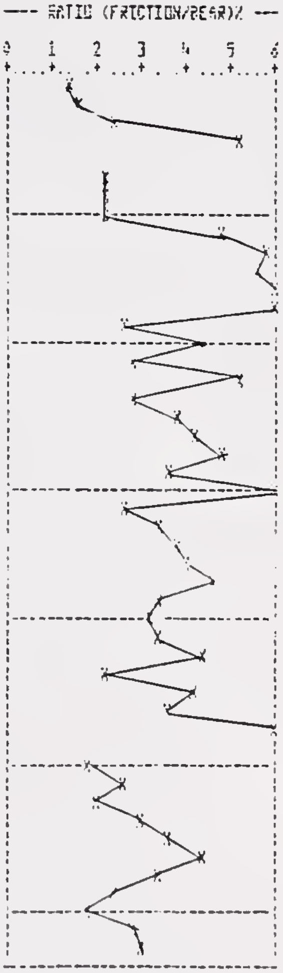
CONE PENETRATION TEST LOGS

```
FILE NAME:      PIEZOBLIDE RESEARCH
FILE NUMBER:    1

DUTCH CONE TEST (CPT)
     SOUNDING NUMBER: CPT 1     PERFORMED IN ACCORD WITH ASTM D-3441, USING FRICTION CONE            TIP
     LOCATION:        LAKE ALICE (NORTH)

     PERFORMED: DATE: 7 JULY 1981
                BY  : ALIREZA BOGHRAT

SURFACE ELEV.=
WATER DEPTH  =  0.90 METER
```

```
     FIELD      FRIC.
     DATA       RATIO

     QC   QC+QF  F/B
    ----  ----   ---
     38.          1.4
     44    52     1.5
     28.   33.    2.4
     10.   25.    5.1
1M   14.   22.    ***
     30.          2.2
     13.   18.    2.2
     31.   37.    2.2
     37.   47     4.7
2M   21.   [illegible]  [illegible]
     33.   [illegible]  [illegible]
     43.   [illegible]  [illegible]
     57.   [illegible]  [illegible]
     43    [illegible]  2.5
3M   25.   41.    4.4
     33.   49.    2.9
     23    37     5.3
     39    57     2.5
     21.   37.    3.6
4M   19.   31.    4.2
     17.   29     4.7
     19.   31.    3.5
     17.   27     7.1
     21.   39.    2.5
5M   31.   39     3.4
     [illegible]
     23.   [illegible]  [illegible]
     23.   37.    [illegible]
     23.   [illegible]  [illegible]
6M   25.   37     [illegible]
     19.   31.    3.5
     21.   31.    4.4
     23.   37.    2.3
     31.   53.    4.3
7M   29.   49.    3.6
     19.   35.    6.5
     19    39.    ***
     31.          1.7
     77.   85.    2.6
8M   75.   105    1.5
     39.   61.    3.9
     29.   47     3.6
     21.   37.    4.3
     64    78.    3.4
9M   66.   94.    2.4
     78.   102.   1.9
     76.   98.    2.3
     86    113.   3.5
    108.   146    ***
```



SOUNDING STOPPED



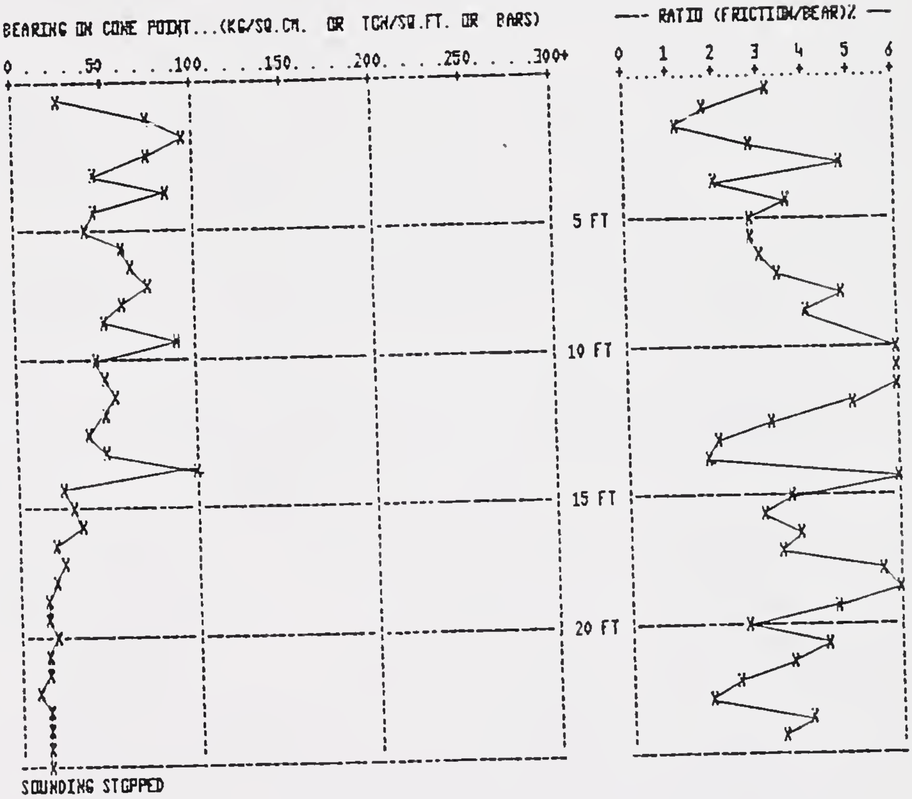
FILE NAME: PIEZOBLIDE RESEARCH
FILE NUMBER: 2

DUTCH CONE TEST (CPT)
SOUNDING NUMBER: CPT 2 PERFORMED IN ACCORD WITH ASTM D-3441, USING FRICTION CONE TIP
LOCATION: LAKE ALICE (EAST)

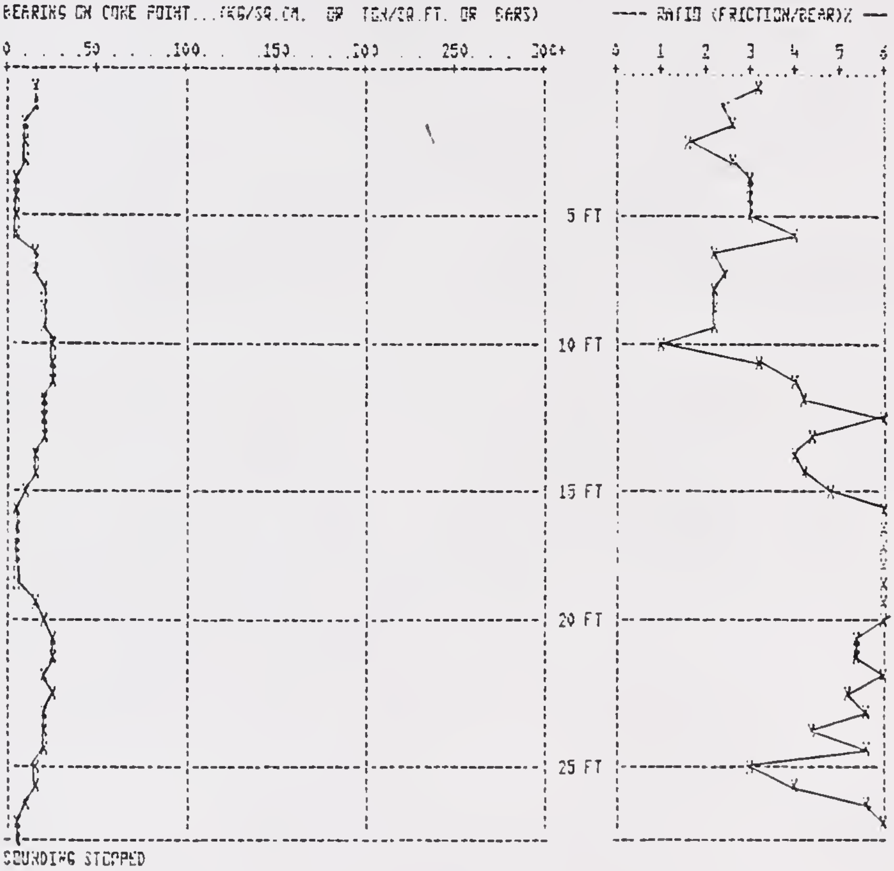
PERFORMED: DATE: 7 JULY 1981
BY : ALIREZA BOGHRAT

SURFACE ELEV.=
WATER DEPTH = 0.45 METER

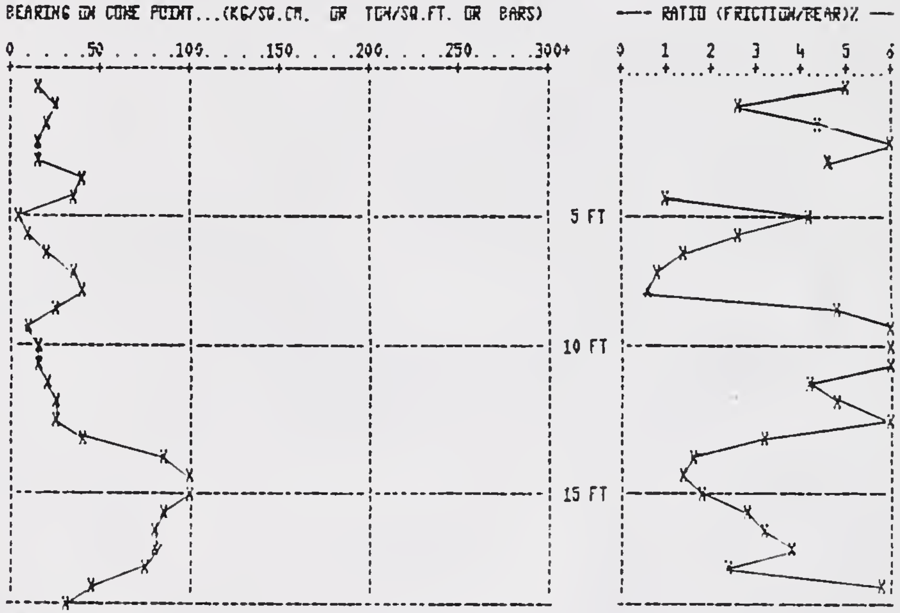
	FIELD DATA		FRIC. RATIO
	QC	QC+SF	F/B
	24.		3.3
	76.	88.	1.7
	96.	116.	1.1
	76.	92.	2.8
1M	44.	76.	4.8
	84.	116.	2.1
	44.	70.	3.6
	39.	63.	2.8
	59.	75.	2.7
2M	63.	87.	3.0
	77.	105.	3.5
	61.	101.	4.8
	51.	95.	4.9
	91.	121.	***
3M	45.		6.7
	49.	107.	7.1
	57.	109.	6.3
	51.	105.	5.0
	41.	79.	3.3
4M	49.	69.	1.9
	101.	115.	1.7
	27.	53.	***
	29.	69.	3.7
	35.	51.	3.1
5M	21.	37.	3.8
	23.	35.	3.5
	19.	31.	5.6
	17.	33.	9.4
	17.	41.	4.7
6M	21.	33.	2.5
	15.	23.	4.4
	15.	25.	3.5
	11.	19.	2.4
	15.	19.	1.6
7M	13.	17.	4.9
	15.	23.	3.5
	13.	21.	***



```
FILE NAME:      PIEZOCALIDE RESEARCH
FILE NUMBER:    3

DUTCH CONE TEST (CPT)
     SOUNDING NUMBER: CPT 3     PERFORMED IN ACCORD WITH ASTM D-3441, USING FRICTION CONE           TIP
     LOCATION:          LAKE ALICE (WEST)

     PERFORMED: DATE: 21 JULY 1981
                BY  : ALIREZA BOGHRAT

SURFACE ELEV.=
WATER DEPTH  =  1.79 METER

     FIELD       FRIC.
     DATA        RATIO

     QC   QC+SF  F/R
    ---- -----   ---
     12.         3.3
     16.   24.   2.4
     10.   16.   2.6
      8.   12.   1.6
 1M  10    12.   2.5
      4.    8.   3.0
      4     6.   3.0
      5.    7    3.0
      7     9.   4.1
 2M  13.   17.   2.1
     17    21.   2.4
     19.   25.   2.1
     19.   25.   2.1
     19.   25.   2.1
 3M  25.   31.   1.1
     25.   29.   3.2
     27.   33.   4.0
     19.   35.   4.3
     21.   33.   6.4
 4M  21.   41.   4.5
     17    31.   4.0
     13    23.   4.1
     11.   19.   4.9
      7    15.   ***
 5M   5    13    ***
      3    17    ***
      3.   17.   ***
      5.   13.   ***
     17.   25.   6.2
 6M  19.   35.   7.0
     25    45    5.3
     27.   47.   5.4
     21.   43.   7.5
     23.   47.   5.2
 7M  19    37.   5.5
     21.   37.   4.4
     21.   35.   5.6
     17.   35.   3.1
     13.   21.   4.3
 8M   9    17.   5.7
      7.   19    5.9
      7.   17    ***
```



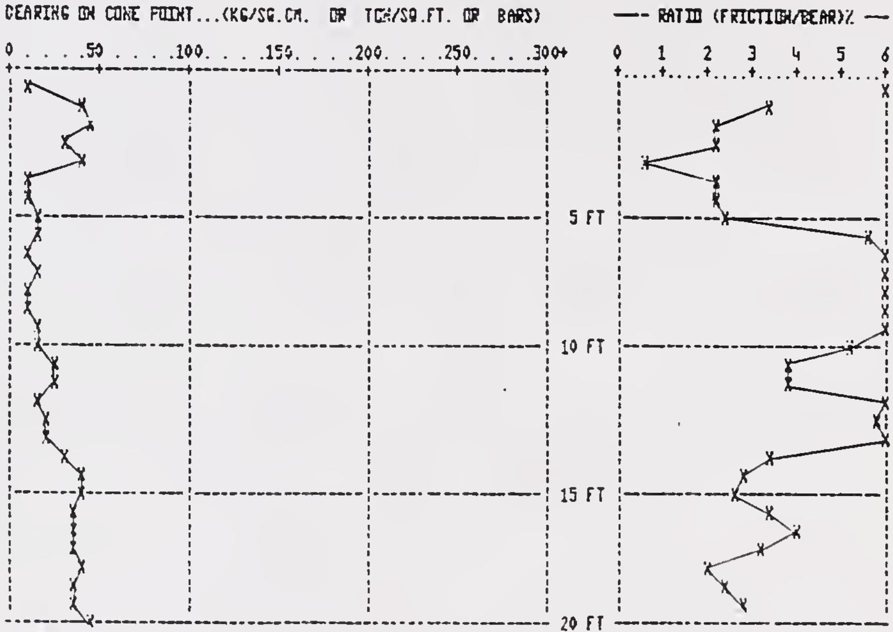
```
FILE NAME:      PIEZOBLADE RESEARCH
FILE NUMBER:    4

DUTCH CONE TEST (CPT)
     SOUNDING NUMBER: CPT 4     PERFORMED IN ACCORD WITH ASTM D-3441, USING FRICTION CONE            TIP
     LOCATION:        LAKE WAUBURG (NORTH WEST)

     PERFORMED: DATE: 15 AUGUST 1981
                BY  : ALIREZA ROCHRAT

SURFACE ELEV.=
WATER DEPTH  =  0.85 METER

    FIELD     FRIC.
    DATA      RATIO

    QC   QC+SF  F/R
   ----  -----  ---
    16.         4.9
    26.    39.  2.5
    18.    29.  4.4
    14.    26.  ***
1M  14.    42.  4.6
    42.    52.  ***
    36.         1.1
     7.    13.  4.1
    11.    15.  2.5
2M  19.    23.  1.4
    37.    41.  0.7
    41.    45.  0.7
    25.    29.  4.9
     9.    27.  9.2
3M  13.    25.  6.3
    15.    27.  7.2
    19.    35.  4.3
    25.    37.  4.8
    23.    41.  8.6
4M  41.    71.  3.3
    85.   105.  1.6
    99.   119.  1.5
    99.   121.  1.9
    83.   111.  2.7
5M  81.   115.  3.1
    81.   119.  3.8
    75.   121.  2.5
    43.    71.  5.9
    31.    69.  ***
```



SOUNDING STOPPED

```
FILE NAME:      PIEZOBLADE RESEARCH
FILE NUMBER:    5

DUTCH CONE TEST (CPT)
     SOUNDING NUMBER: CPT 5     PERFORMED IN ACCORD WITH ASTM D-3441, USING FRICTION CONE          TIP
     LOCATION:        LAKE WAUBURG (NORTH WEST)

     PERFORMED: DATE: 15 AUGUST 1981
                BY  : ALIREZA BOGHRAT

SURFACE ELEV.=
WATER DEPTH  =  0.90 METER
```

	FIELD DATA QC	QC+SF	FRIC. RATIO F/B
	12.		***
	40.	62.	3.3
	44.	64.	2.1
	30.	44.	2.2
1M	38.	48.	0.7
	12.	16.	2.1
	12.	16.	2.1
	17.	21.	2.4
	17.	23.	5.6
2M	11.	25.	**.*
	15.	31.	7.3
	11.	27.	8.8
	11.	25.	7.5
	13.	25.	9.5
3M	13.	31.	5.3
	25.	35.	3.8
	25.	39.	3.8
	17.	31.	7.9
	21.	41.	5.6
4M	21.	39.	6.4
	31.	51.	3.5
	39.	55.	2.7
	41.	57.	2.6
	35.	51.	3.4
5M	33.	51.	4.0
	37	57.	3.2
	41	59.	2.0
	33.	45.	2.4
	37.	49.	2.9
6M	45.	61.	***



SOUNDING STOPPED

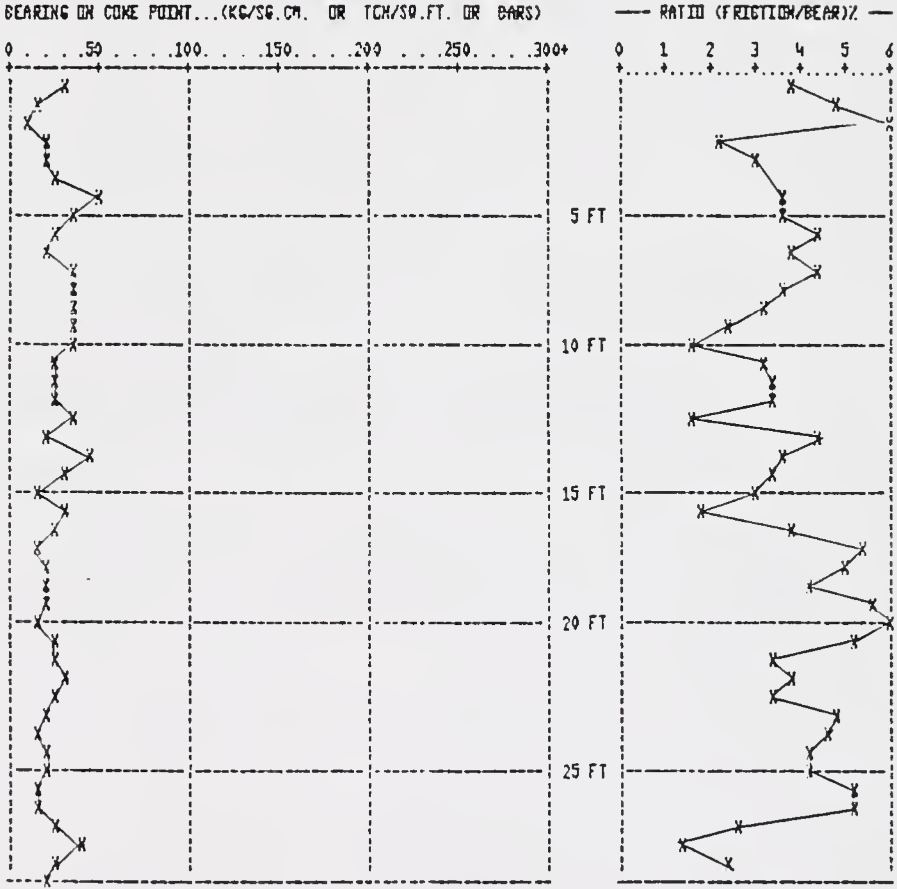
FILE NAME: PIEZOBLADE RESEARCH
FILE NUMBER: 6

DUTCH CONE TEST (CPT)
SOUNDING NUMBER: CPT 6 PERFORMED IN ACCORD WITH ASTM D-3441, USING FRICTION CONE TIP
LOCATION: LAKE ALICE (NORTH)

PERFORMED: DATE: 23 SEPTEMBER 1981
BY : ALIREZA BOGHRAT

SURFACE ELEV.=
WATER DEPTH = 0.90 METER

	FIELD DATA		FRIC. RATIO
	QC	QC+SF	F/B
	32.		3.7
	16.	[illegible]	4.9
	12.	24.	6.5
	18.	30.	2.2
1M	22.	28.	3.0
	24.	34.	***
	48.		3.6
	33.	59.	3.7
	25.	43.	4.4
2M	21.	37.	3.9
	37.	49.	4.4
	33.	57.	3.7
	33.	51.	3.3
	35.	51.	2.3
3M	35.	47.	1.5
	25.	33.	3.2
	27.	39.	3.5
	27.	41.	3.5
	35.	49.	1.5
4M	21.	29.	4.5
	45.	59.	3.6
	31.	55.	3.5
	13.	29.	3.1
	29.	35.	1.8
5M	25.	33.	3.7
	17.	31.	5.5
	21.	35.	5.1
	19.	35.	4.2
	19.	31.	5.6
6M	15.	31.	7.1
	23.	39.	5.2
	27.	45.	3.4
	31.	45.	3.8
	23.	41.	3.4
7M	19.	31.	4.3
	17.	31.	4.6
	19.	31.	4.1
	19.	31.	4.1
	15.	27.	5.2
8M	15.	27.	5.2
	25.	37.	2.6
	39.	49.	1.4
	27.	35.	2.4
	22.	32.	***



SOUNDING STOPPED

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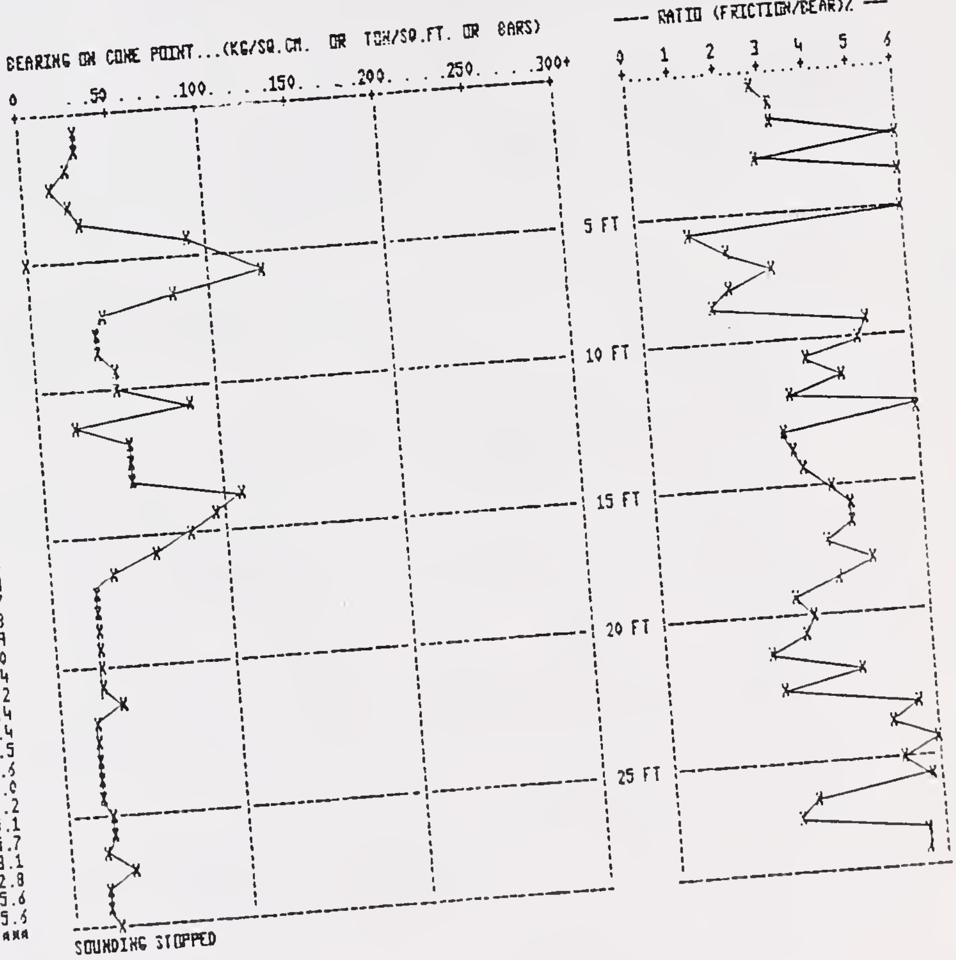
FILE NAME:      PIEZOBLADE RESEARCH
FILE NUMBER:    7
                                                                                       TIP
DUTCH CONE TEST (CPT)                 PERFORMED IN ACCORD WITH ASTM D-3441, USING FRICTION CONE
     SOUNDING NUMBER: CPT 7
     LOCATION:        LAKE ALICE (NORTH)

     PERFORMED: DATE: 23 SEPTEMBER 1981
                BY  : ALIREZA BOCHRAT

   SURFACE ELEV.=
   WATER DEPTH  =  0.90 METER

  FIELD        FRIC.
  DATA         RATIO

   QC   QC+SF   F/B
  ---   -----   ---
   32.          2.9
   28.    42.   3.3
   24.    38.   3.3
   16.    28.   7.3
1M 24.    42.   2.7
   30.    40.   7.9
   88.   124.   0.0
    1.     1.   ***
  131.   157.   1.2
2M 81.   105.   2.0
   41.    65.   3.0
 - 33.    51.   2.0
   33.    43.   1.6
   43.    51.   5.0
3M 47.    79.   4.9
   87.   121.   3.7
   21.    69.   4.5
   49.    63.   3.3
   51.    75.   7.4
4M 49.   105.   3.0
  109.   131.   3.2
   93.   145.   3.4
   81.   129.   4.0
   61.   109.   4.4
5M 37.    77.   4.3
   25.    49.   3.7
   25.    39.   4.8
   27.    45.   3.9
   27.    43.   3.0
6M 27.    39.   3.4
   25.    39.   3.2
   33.    45.   2.4
   21.    33.   4.4
   21.    35.   2.5
7M 21.    29.   5.6
   21.    39.   5.0
   19.    35.   6.2
   23.    41.   5.1
   23.    41.   5.7
8M 21.    41.   3.1
   33.    43.   2.8
   21.    35.   5.6
   21.    39.   5.6
   24.    42.   ***
```



```
FILE NAME:      PIEZOBLADE RESEARCH
FILE NUMBER:    8

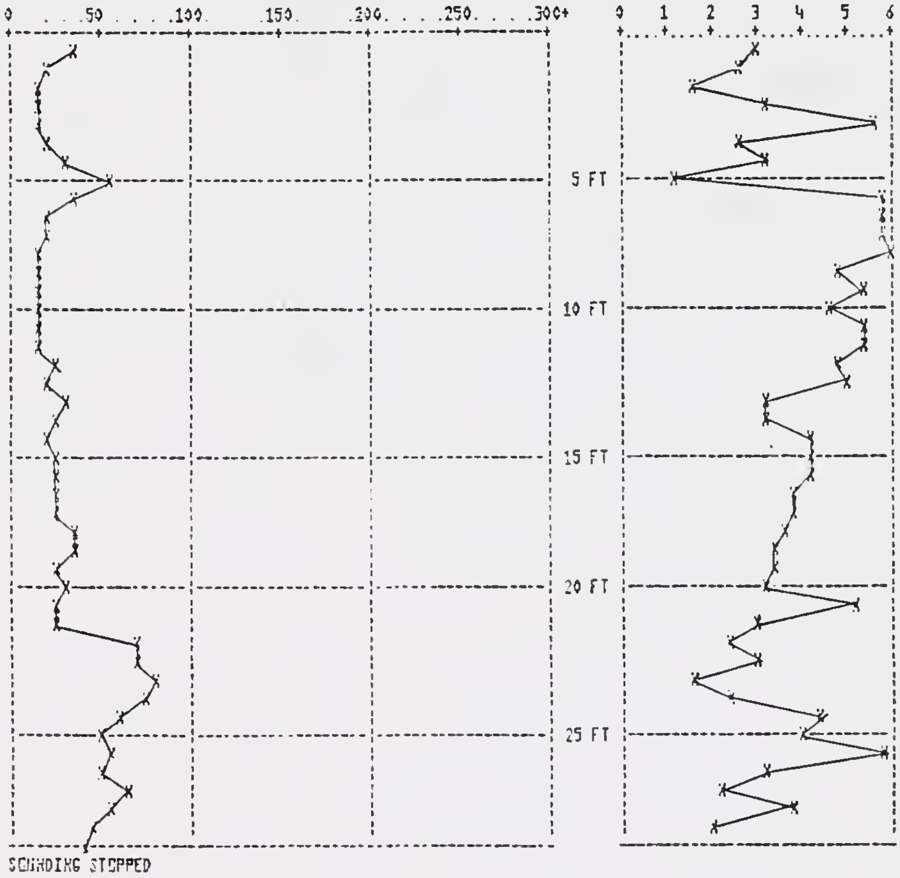
DUTCH CONE TEST (CPT)
     SOUNDING NUMBER: CPT 8     PERFORMED IN ACCORD WITH ASTM D-3441, USING FRICTION CONE           TIP
     LOCATION:        LAKE ALICE (NORTH)

     PERFORMED: DATE: 23 SEPTEMBER 1981
                BY  : ALIREZA BOGHRAT

SURFACE ELEV.=
WATER DEPTH  =  0.80 METER

    FIELD        FRIC.
    DATA         RATIO    BEARING ON CONE POINT...(KG/SQ.CM.  OR  TON/SQ.FT. OR  BARS)        ---- RATIO (FRICTION/BEAR)% ---

     QC   QC+SF   F/R
    ----  -----   ---
     36.           2.9
     20.    36.    2.5
     16.    24.    1.4
     16.    20.    3.3
1M   16.    24.    5.7
     20.    34.    2.6
     32.    40.    3.3
     57.    73.    1.2
     33.    41.    5.7
2M   21.    49     5.8
     21.    39.    5.5
     13     31     9.5
     17.    35.    4.9
     15.    27.    5.5
3M   15.    27.    4.5
     15.    25.    5.4
     15.    27.    5.4
     25.    37.    4.8
     19.    37.    5.0
4M   29.    43.    3.2
     25.    39.    3.2
     19.    31.    4.2
     25.    37.    4.3
     25.    41.    4.3
5M   25     41.    3.7
     25     39     3.7
     33.    47.    3.4
     35.    53.    3.4
     27.    45.    3.4
6M   29.    43.    3.2
     23.    37.    5.2
     27.    45.    2.5
     69.    81.    2.3
     71.    95.    3.0
7M   91.   113.    1.5
     77     97.    2.4
     61.    69.    4.4
     49     69.    4.1
     55.    95.    5.9
8M   49.    97.    3.2
     63.    87.    2.1
     57.    77.    3.7
     45.    77.    2.1
     38.    52.   [illegible]
```



```
FILE NAME:      PIEZOBLADE RESEARCH
FILE NUMBER:    9

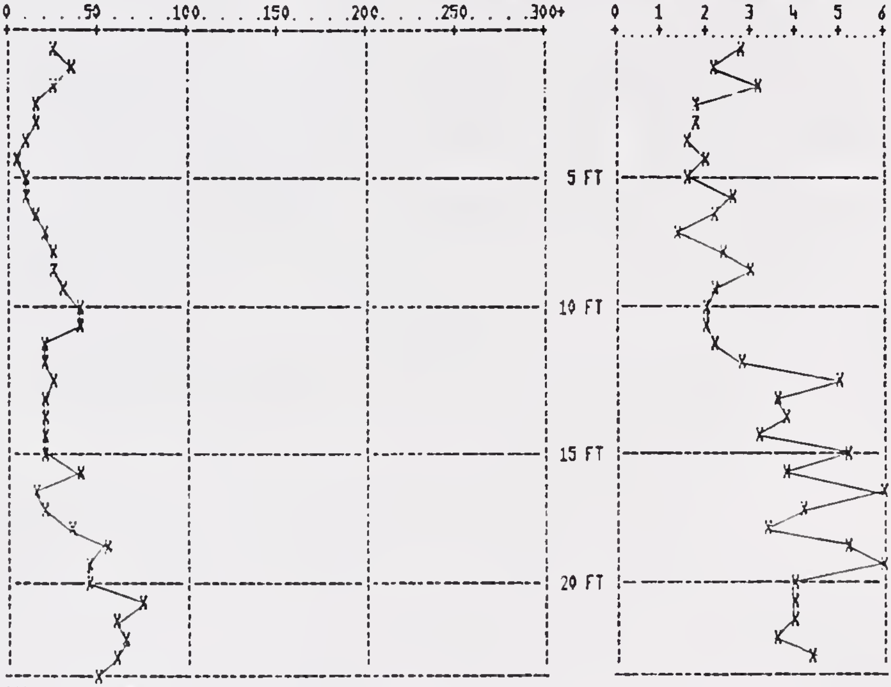
DUTCH CONE TEST (CPT)
     SOUNDING NUMBER: CPT 9     PERFORMED IN ACCORD WITH ASTM D-3441, USING FRICTION CONE           TIP
     LOCATION:        LAKE ALICE (WEST)

     PERFORMED: DATE: 24 SEPTEMBER 1981
                BY  : ALIREZA BOGHRAT

SURFACE ELEV.=
WATER DEPTH  =  1.50 METER

    FIELD      FRIC.
    DATA       RATIO   BEARING ON CONE POINT...(KG/SQ.CM.  OR  TON/SQ.FT.- OR  BARS)          --- RATIO (FRICTION/BEAR)% ---

    QC   QC+SF  F/B
    ---  -----  ---
    24.         2.7
    36.   46.   2.2
    24.   36.   3.3
    14.   26.   1.9
1M  14.   18.   1.9
     8.   12.   1.6
     6.    8.   2.1
     9.   11.   1.6
    11.   13.   2.5
2M  13.   17.   2.1
    19    23.   1.4
    23.   27.   2.4
    23.   31.   2.9
    31.   41.   2.2
3M  39.   49.   2.1
    39.   51.   2.1
    19.   31.   2.1
    19.   25.   2.8
    27.   35.   5.0
4M  19.   39.   3.5
    21.   31.   3.8
    21.   33.   3.2
    21.   31.   5.1
    39.   55.   3.8
5M  15.   37.   6.2
    19.   33.   4.2
    35.   47.   3.4
    53.   71.   5.3
    47    89.   5.9
6M  47.   89.   4.0
    73.  101.   4.0
    61.  105.   3.9
    65.  101.   3.7
    61.   97.   4.4
7M  49.   89.   ***
```



SOUNDING STOPPED

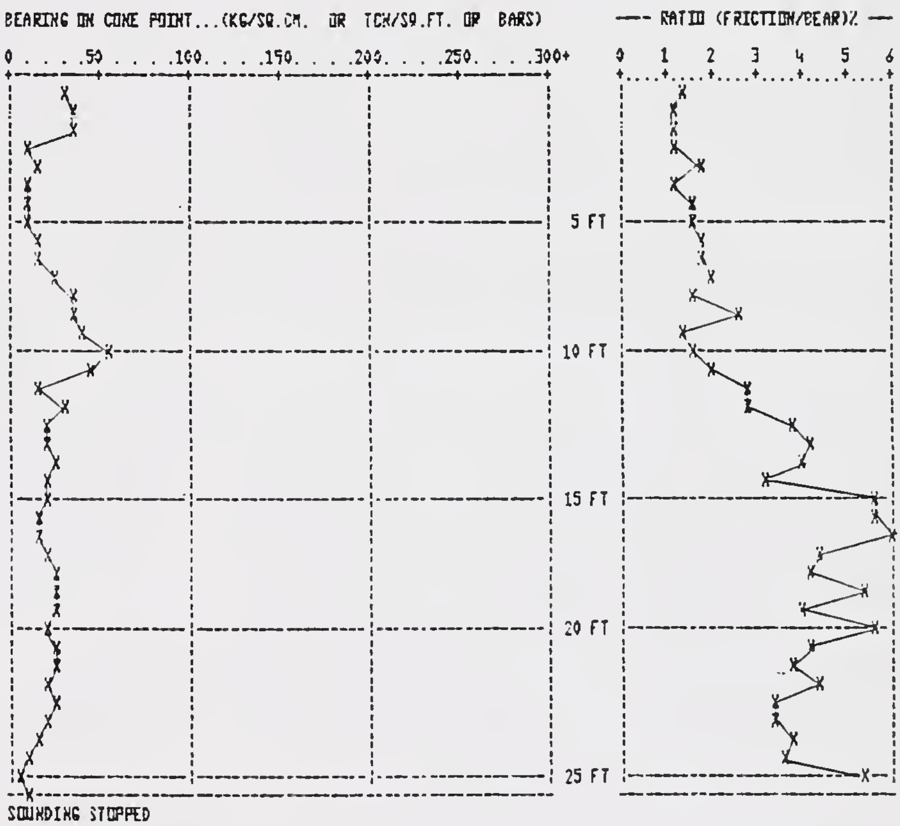
FILE NAME: PIEZOBLADE RESEARCH
FILE NUMBER: 10

DUTCH CONE TEST (CPT)
SOUNDING NUMBER: CPT 10 PERFORMED IN ACCORD WITH ASTM D-3441, USING FRICTION CONE TIP
LOCATION: LAKE ALICE (WEST)

PERFORMED: DATE: 25 SEPTEMBER 1981
BY : ALIREZA BOGHRAT

SURFACE ELEV.=
WATER DEPTH = 1.90 METER

	FIELD DATA		FRIC. RATIO
	QC	QC+SF	F/B
	30.		1.3
	36.	42.	1.1
	34.	40.	1.2
	10.	16.	1.3
1M	14.	16.	1.9
	10.	14.	1.3
	8.	10.	1.6
	9.	11.	1.6
	15.	17.	1.8
2M	15.	19.	1.8
	27.	31.	2.0
	33.	41.	1.6
	37.	45.	2.5
	41.	55.	1.3
3M	53.	61.	1.5
	45.	57.	2.1
	15.	29.	2.7
	29.	35.	2.8
	21.	33.	3.8
4M	19.	31.	4.2
	23.	35.	4.1
	21.	35.	3.2
	19.	29.	5.6
	17.	33.	5.5
5M	13.	27.	6.2
	21.	33.	4.4
	25.	39.	4.3
	25.	41.	5.3
	27.	47.	3.9
6M	19.	35.	5.6
	25.	41.	4.2
	25.	41.	3.7
	21.	35.	4.4
	23.	37.	3.4
7M	19.	31.	3.5
	17.	27.	3.9
	11.	21.	3.5
	7.	13.	5.4
	9.	15.	***



```
FILE NAME:      PIEZOBLADE RESEARCH
FILE NUMBER:    11

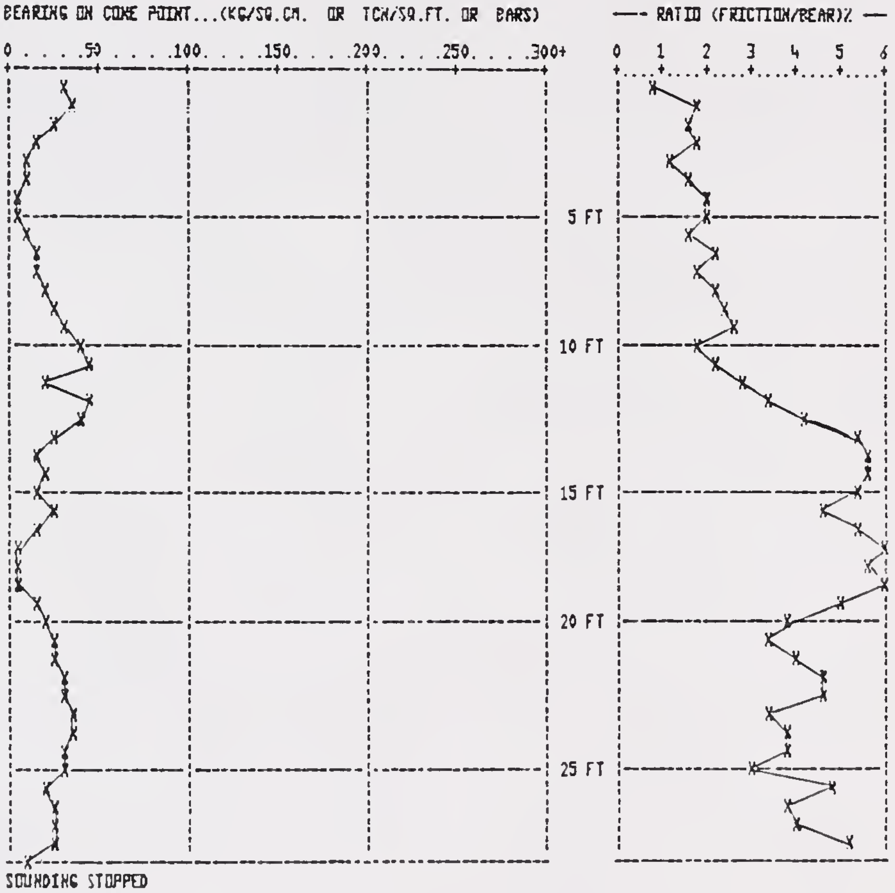
DUTCH CONE TEST (CPT)
     SOUNDING NUMBER: CPT 11    PERFORMED IN ACCORD WITH ASTM D-3441, USING FRICTION CONE           TIP
     LOCATION:          LAKE ALICE (WEST)

     PERFORMED: DATE: 24 SEPTEMBER 1981
                BY  : ALIREZA BOGHRAT

SURFACE ELEV.=
WATER DEPTH  =  1.40 METER
```

```
      FIELD      FRIC.
      DATA       RATIO

      QC   QC+SF  F/R
      ---  ----   --
      30.         0.9
      34.   40.   1.8
      24.   34.   1.6
      14.   20.   1.9
1M    10.   14.   1.3
       8.   10.   1.6
       6.    8.   2.1
       7.    9.   2.1
       9.   11.   1.6
2M    13.   15.   2.1
      15.   19.   1.6
      19.   23.   2.1
      23.   29.   2.4
      31.   39.   2.6
3M    39.   51.   1.7
      43.   53.   2.2
      19.   33.   2.6
      47.   55.   3.4
      39.   63.   4.1
4M    25.   49.   5.4
      17.   37.   5.5
      19.   33.   5.6
      15.   31.   5.4
      23.   35.   4.6
5M    15.   31.   5.3
       7.   19.   7.6
       7.   15.   5.7
       7.   13.   7.6
      13.   21.   5.1
6M    21.   31.   3.8
      23.   35.   3.5
      27.   39.   3.9
      29.   45.   4.6
      31.   51.   4.7
7M    35.   57.   3.4
      35.   53.   3.3
      31.   51.   3.8
      31.   49.   3.0
      19.   33.   4.8
8M    27.   41.   3.9
      23.   39.   4.0
      23.   37.   5.1
      11.   29.   ***
```



```
FILE NAME:      PIEZOBLIDE RESEARCH
FILE NUMBER:    12

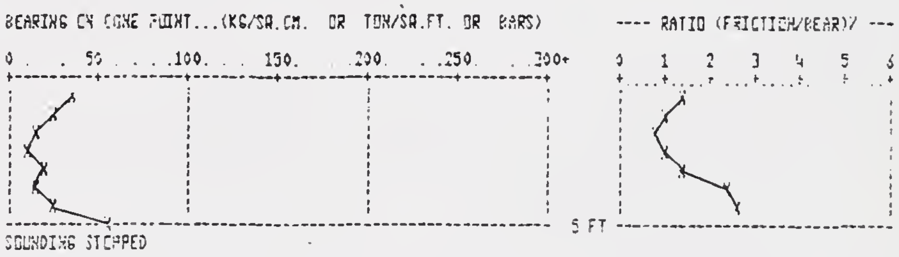
DUTCH CONE TEST (CPT)
     SOUNDING NUMBER: CPT 12    PERFORMED IN ACCORD WITH ASTM D-3441, USING FRICTION CONE           TIP
     LOCATION:        LAKE ALICE (NORTH)

     PERFORMED: DATE: 16 JUNE 1982
                BY  : ALIREZA BOGHRAT

SURFACE ELEV.=
WATER DEPTH  =  0.90 METER

    FIELD      FRIC.
    DATA       RATIO

    QC   QC+SF  F/R
   ---- -----  ---
    36.         1.3
    26.   34.   1.0
    16.   20.   0.8
    12.   14.   1.1
1M  18.   20.   1.4
    16.   20.   2.4
    26    32.   2.5
    55.   65.   ***
```



APPENDIX D

CONSOLIDATION TEST RESULTS

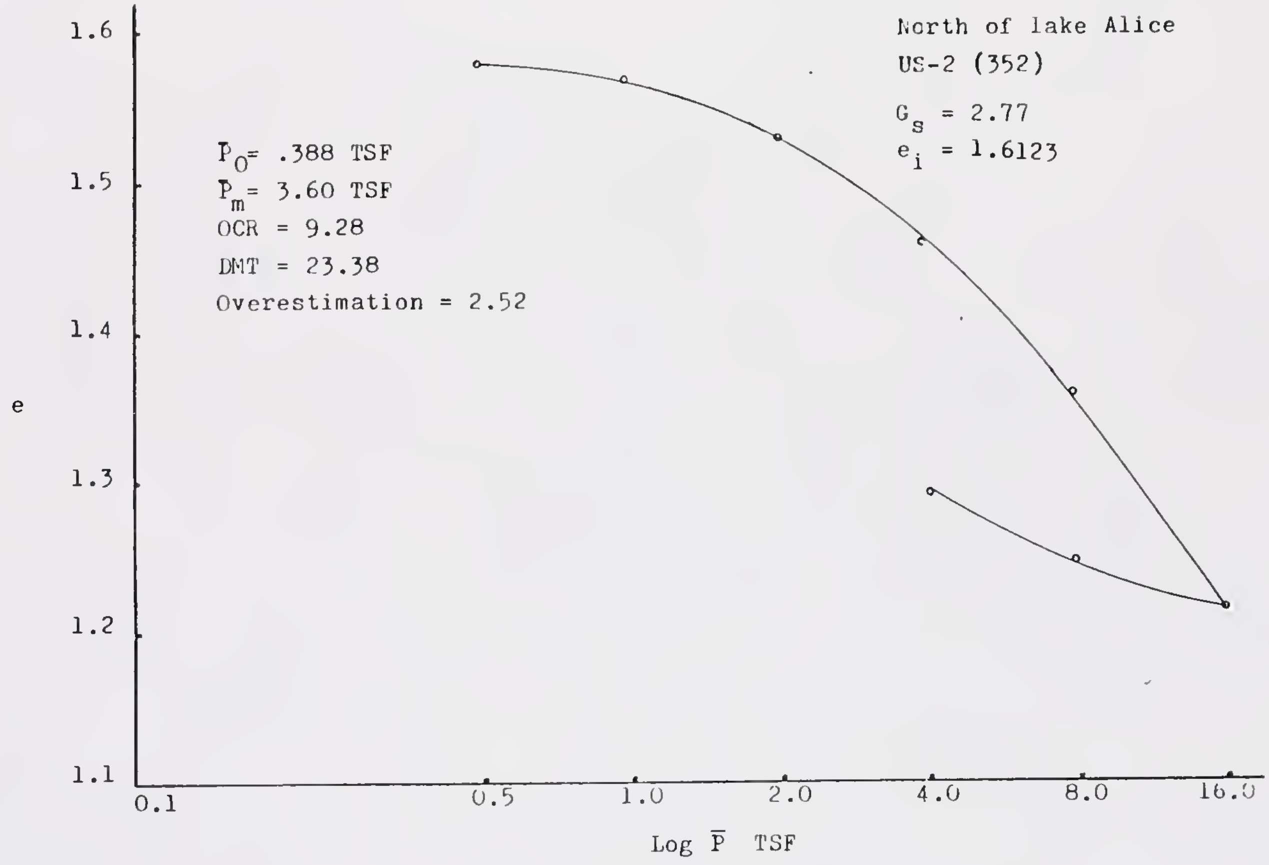


Figure D-1. e - log $\bar{P}$ plot from US-2 (352)

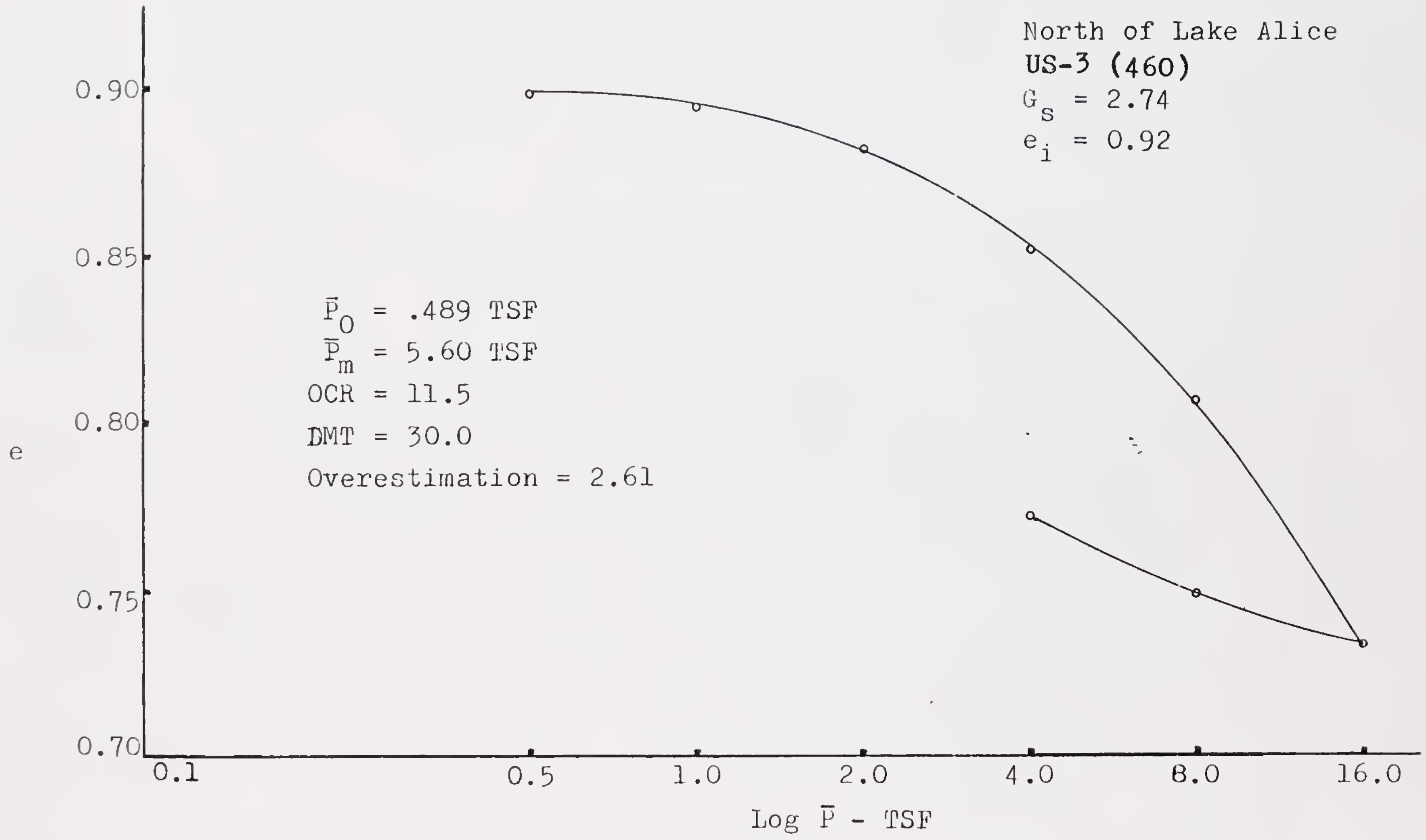


Figure D-2. e - log $\overline{P}$ plot from US-3 (460)

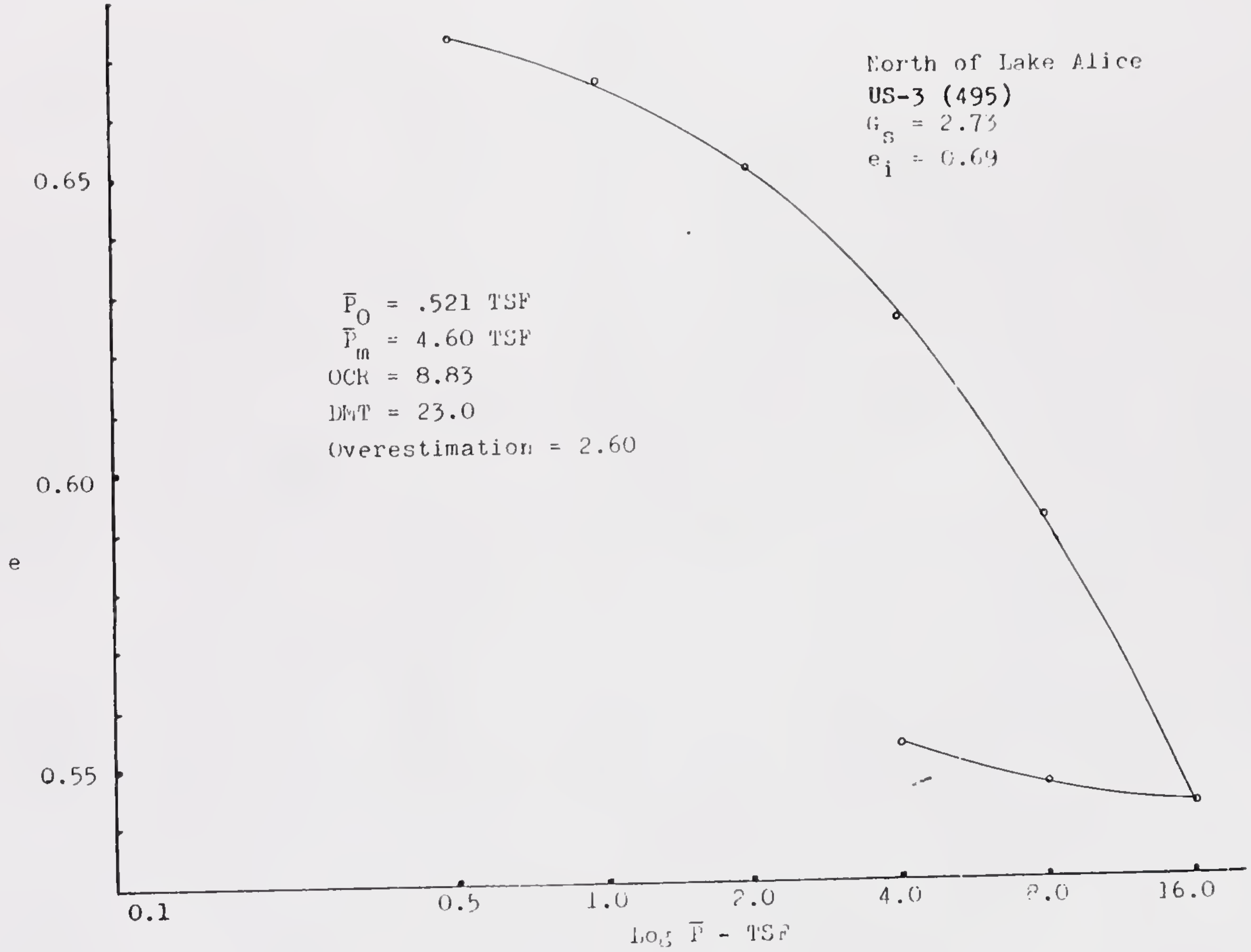


Figure D-3. e - log $\overline{P}$ plot from US-3 (495)

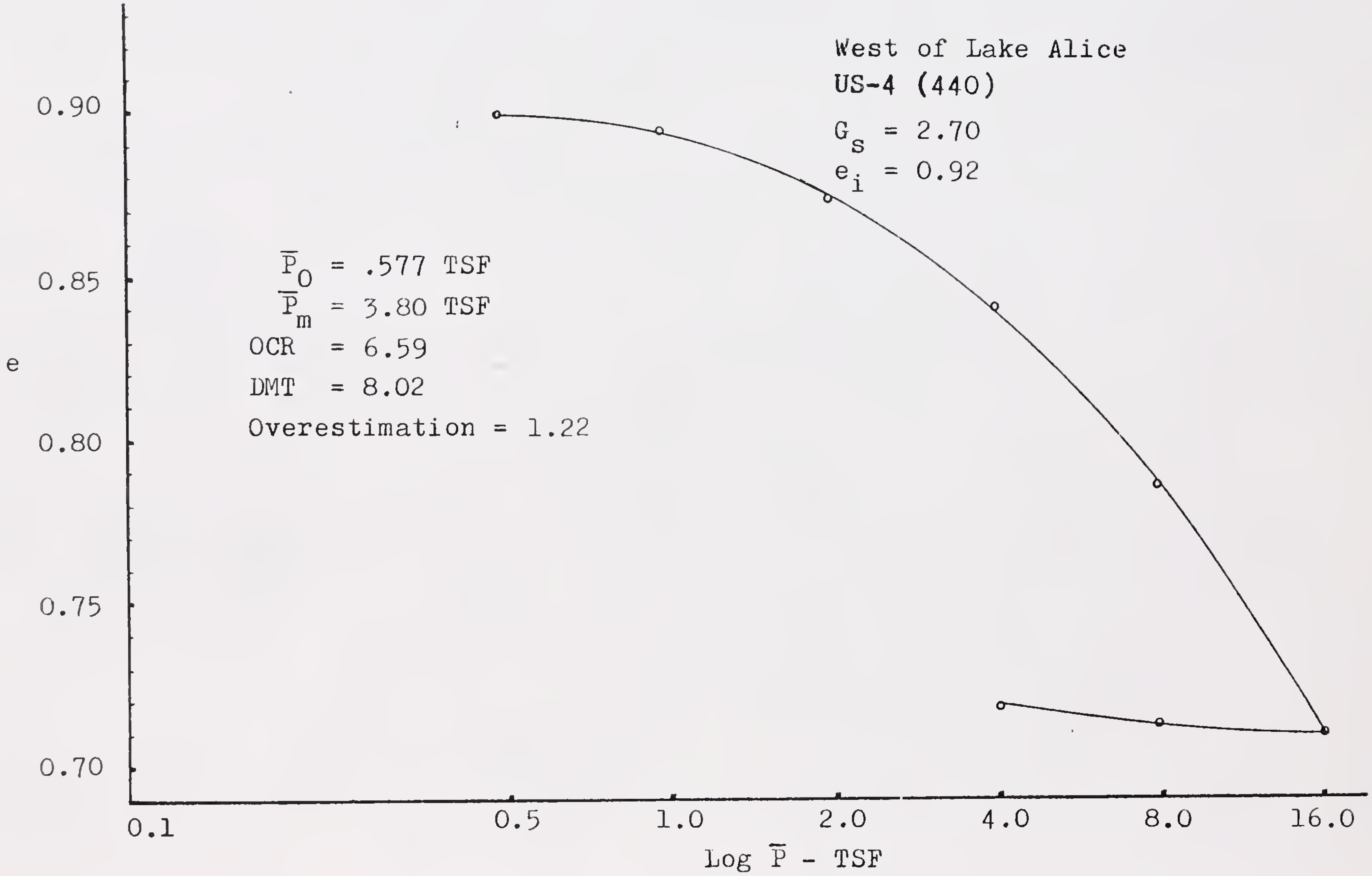


Figure D-4. e - log $\overline{P}$ plot from US-4 (440)

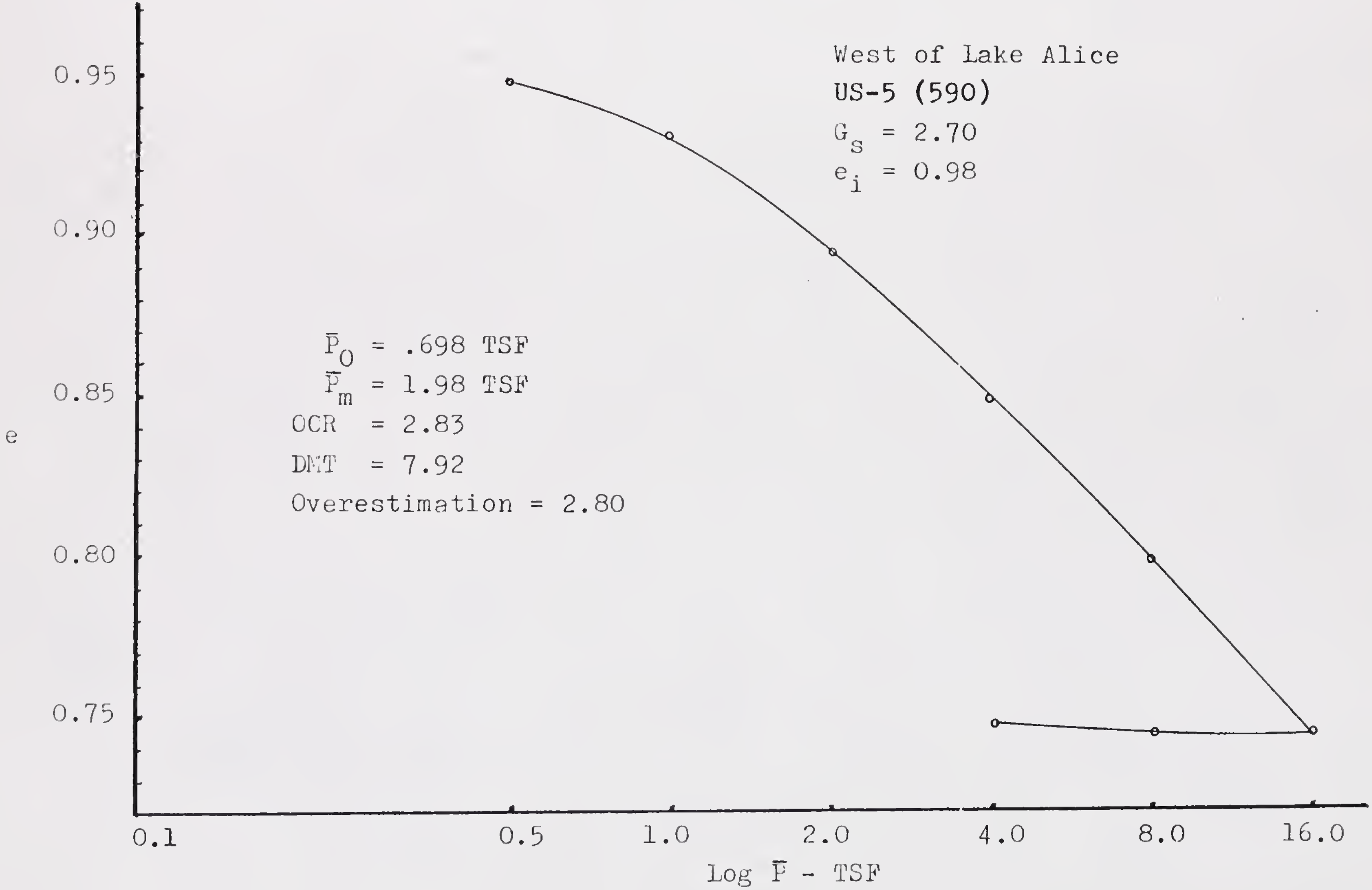


Figure D-5. e - log $\overline{P}$ plot from US-5 (590)

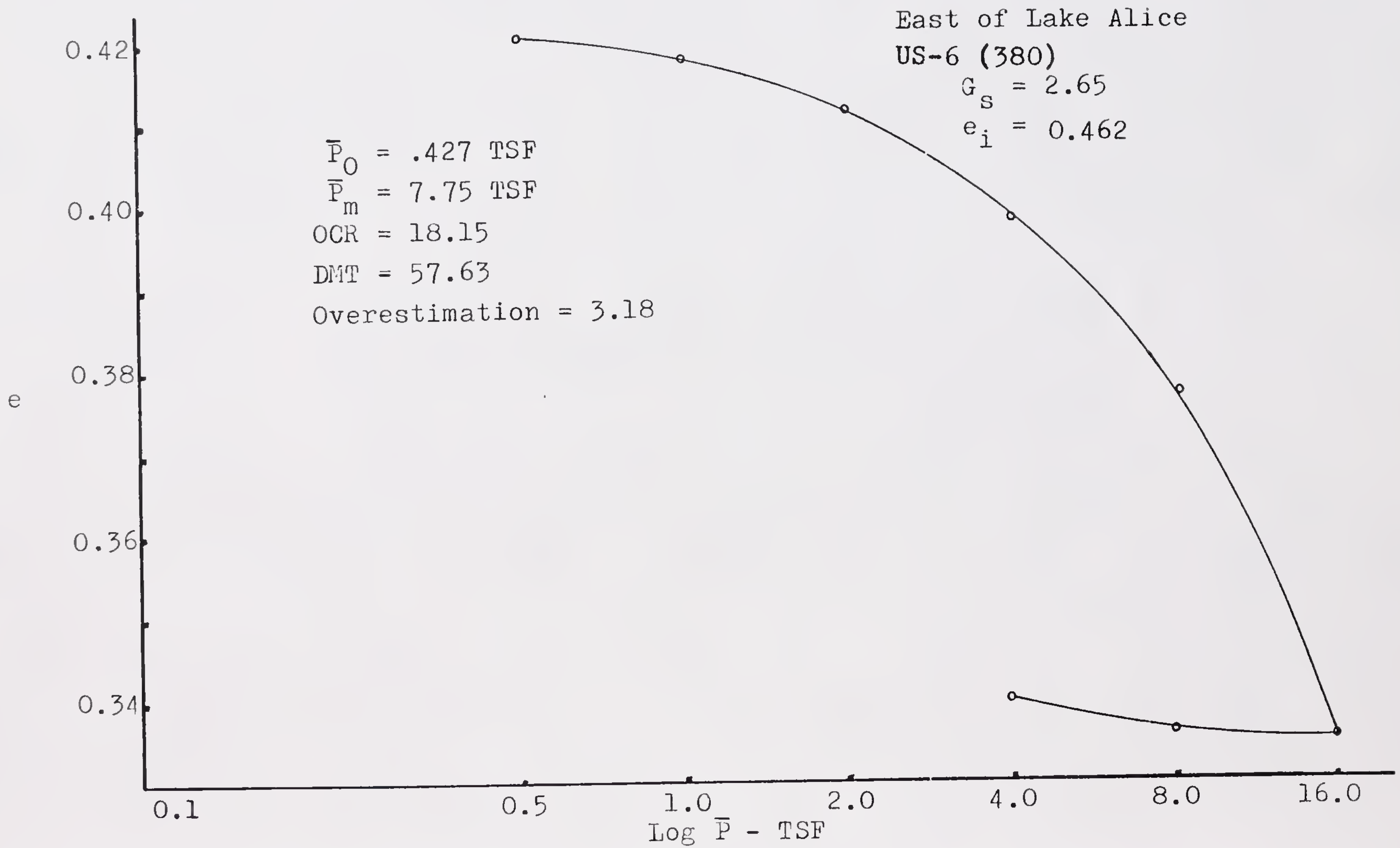


Figure D-6. e - log $\overline{P}$ plot from US-6 (380)

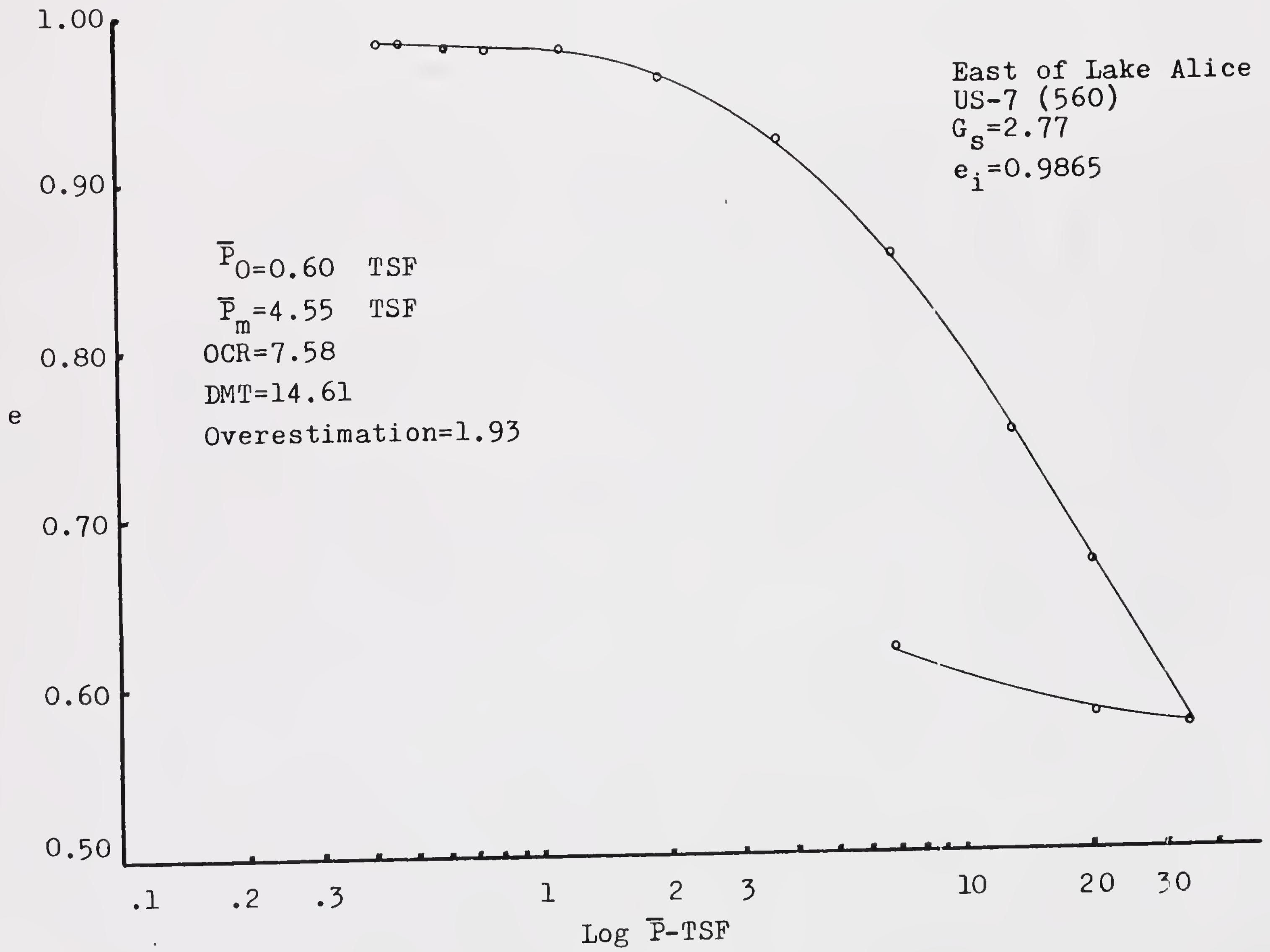


Figure D-7. e - log $\overline{P}$ plot from US-7 (560)

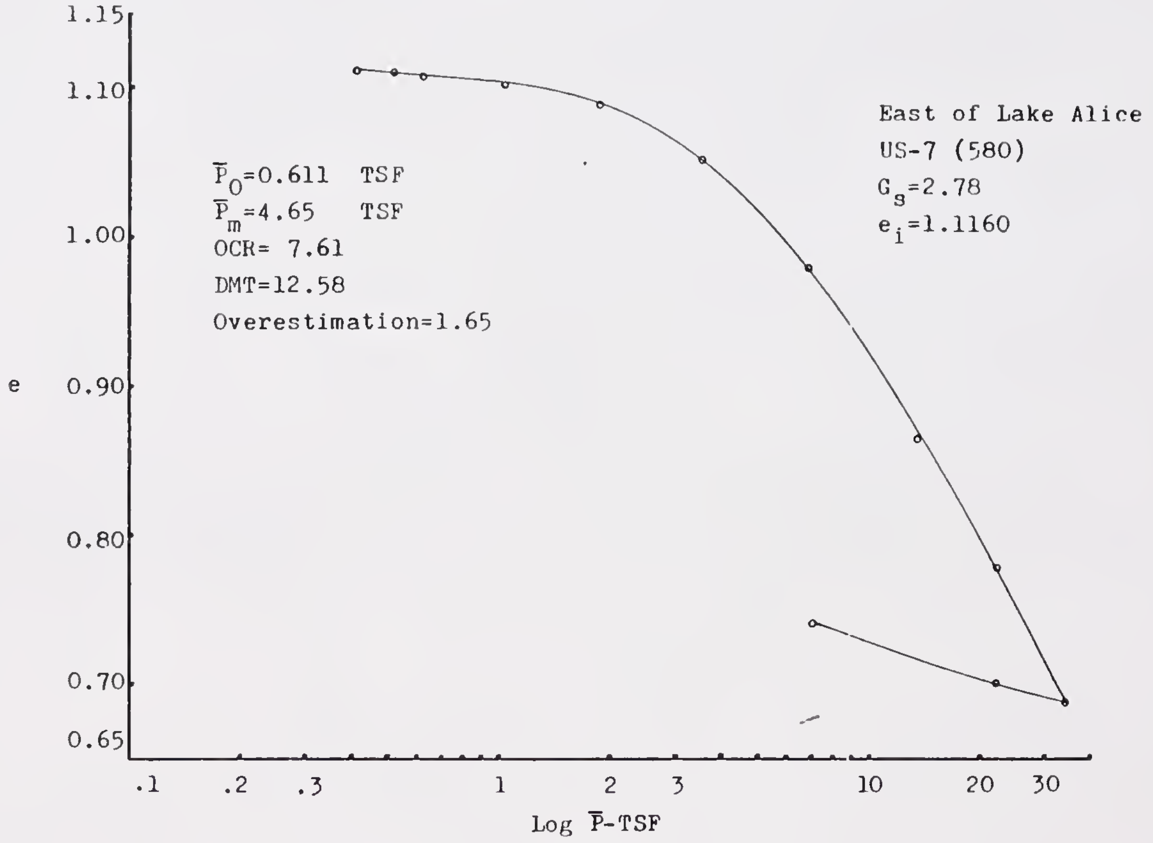


Figure D-8. e - log $\overline{P}$ plot from US-7 (580)

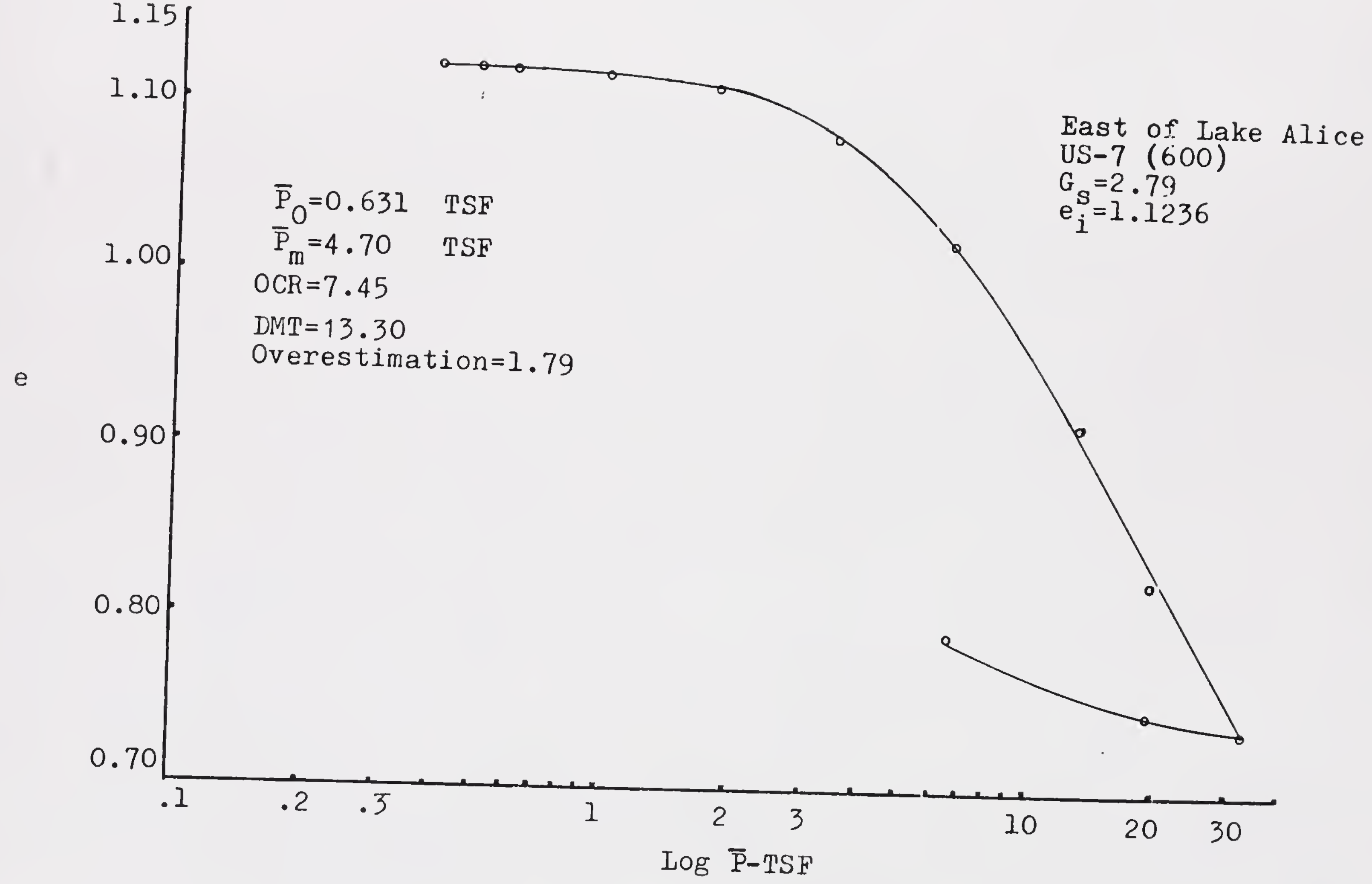


Figure D-9. e - log $\overline{P}$ plot from US-7 (600)

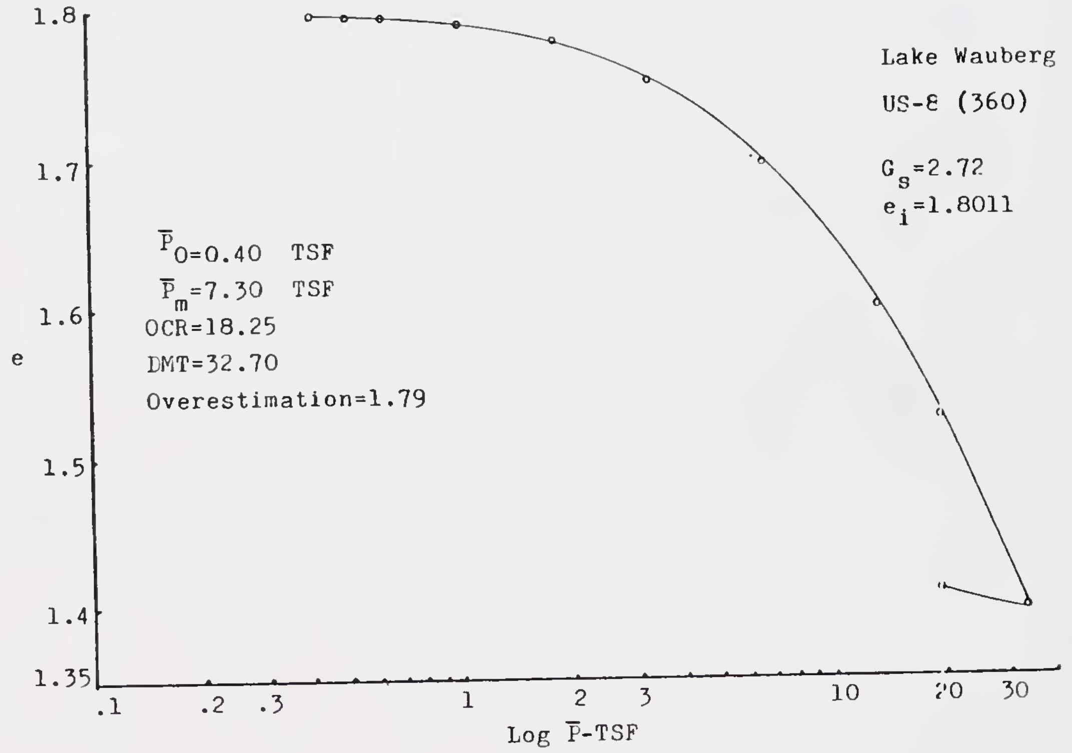


Figure D-10. e - log $\overline{P}$ plot from US-8 (360)

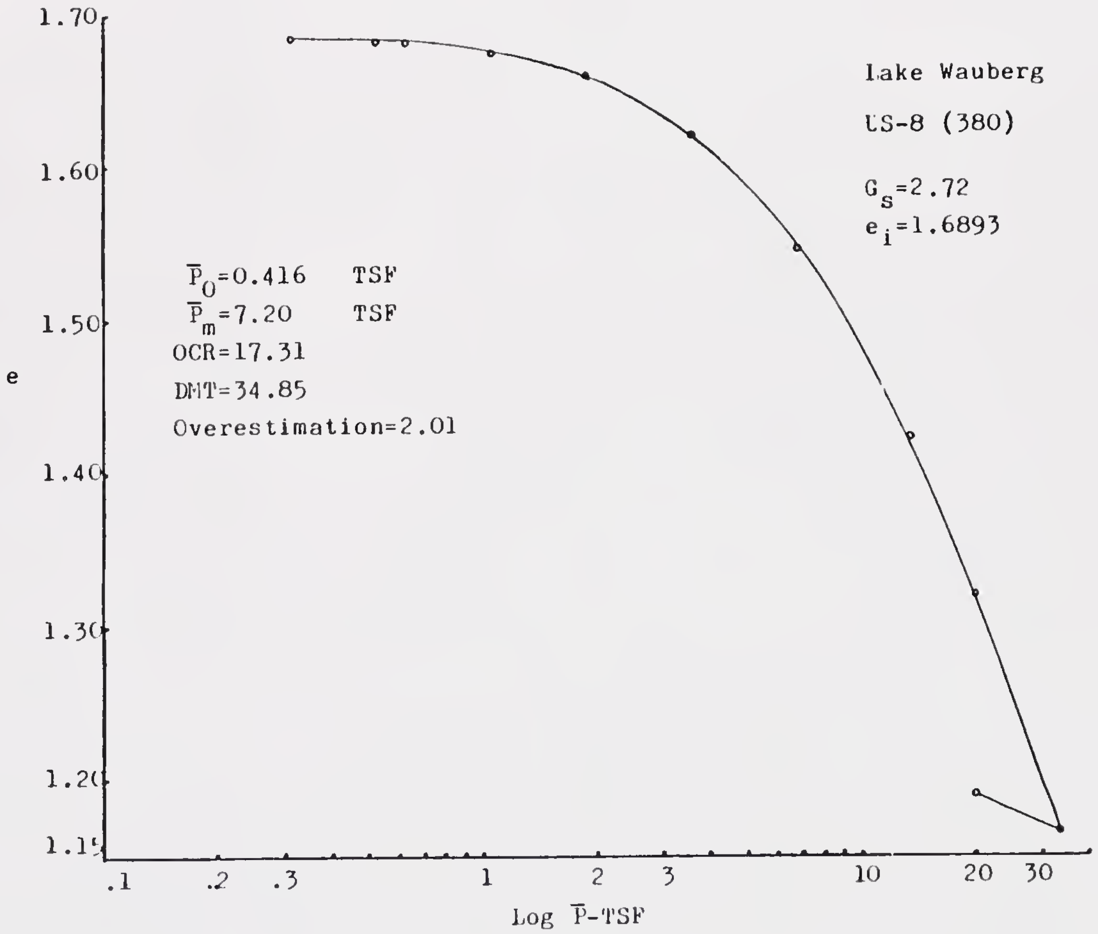


Figure D-11. e - log $\overline{P}$ plot from US-8 (380)

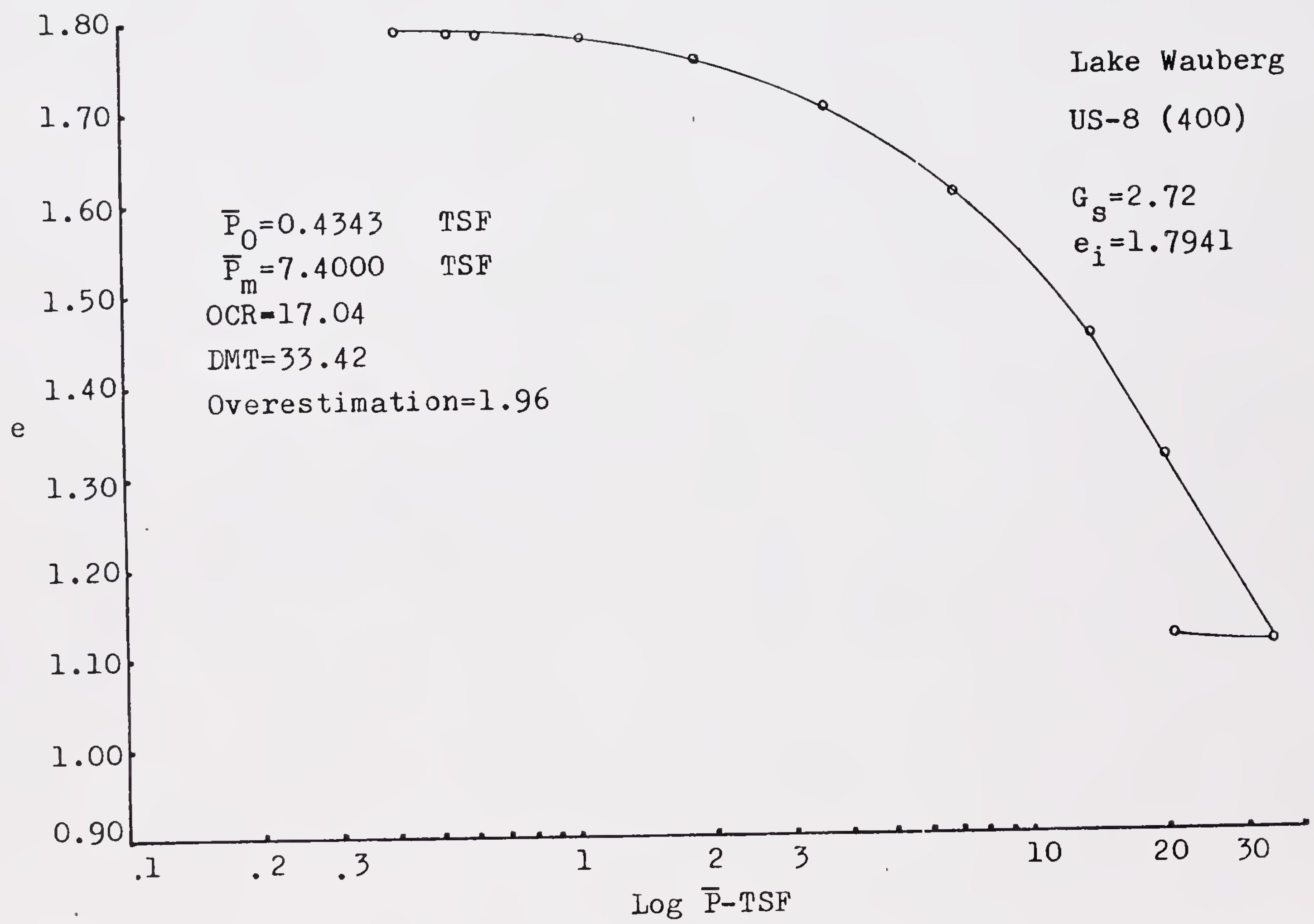


Figure D-12. e - log $\overline{P}$ plot from US-8 (400)

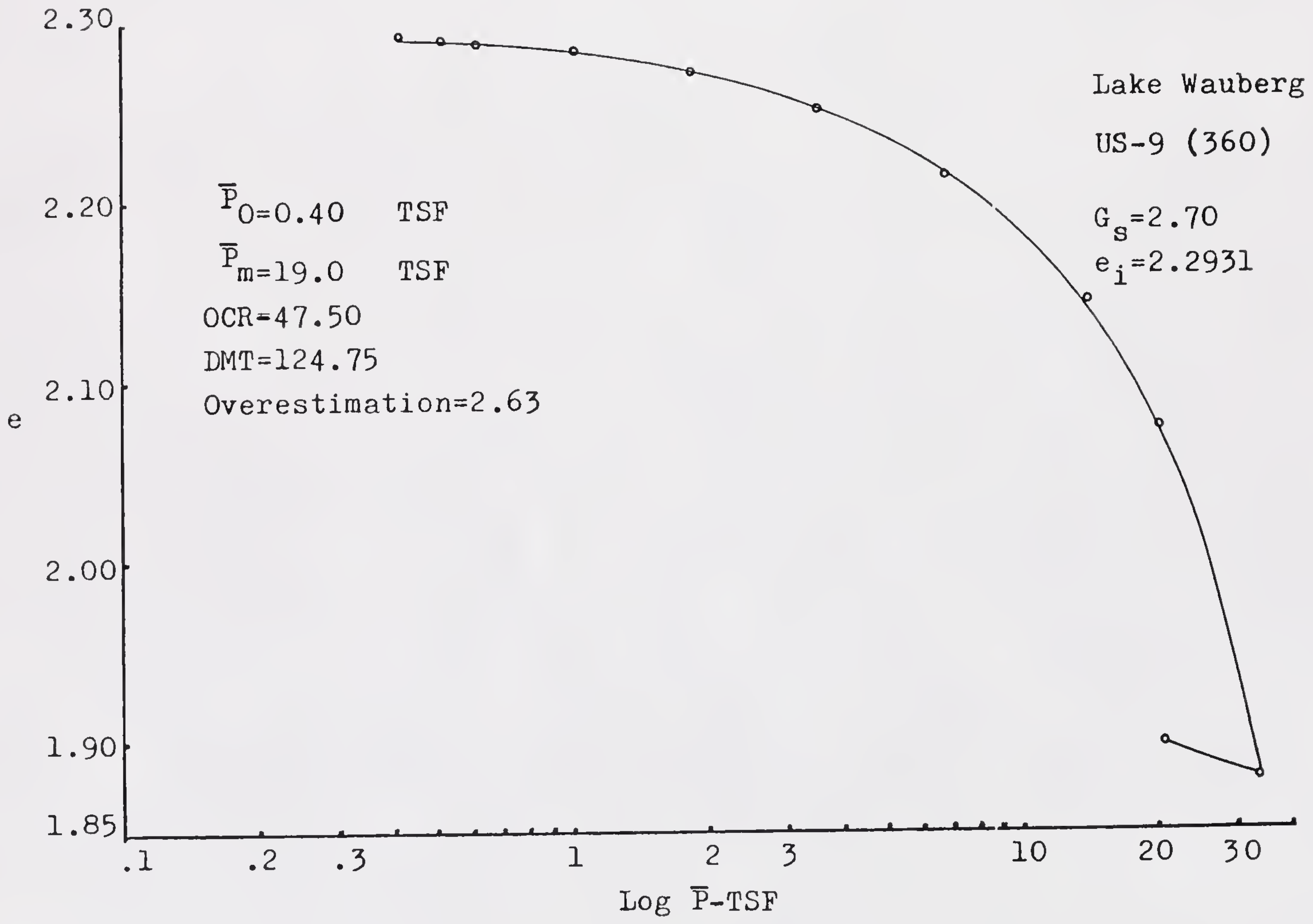


Figure D-13. e - log $\overline{P}$ plot from US-9 (360)

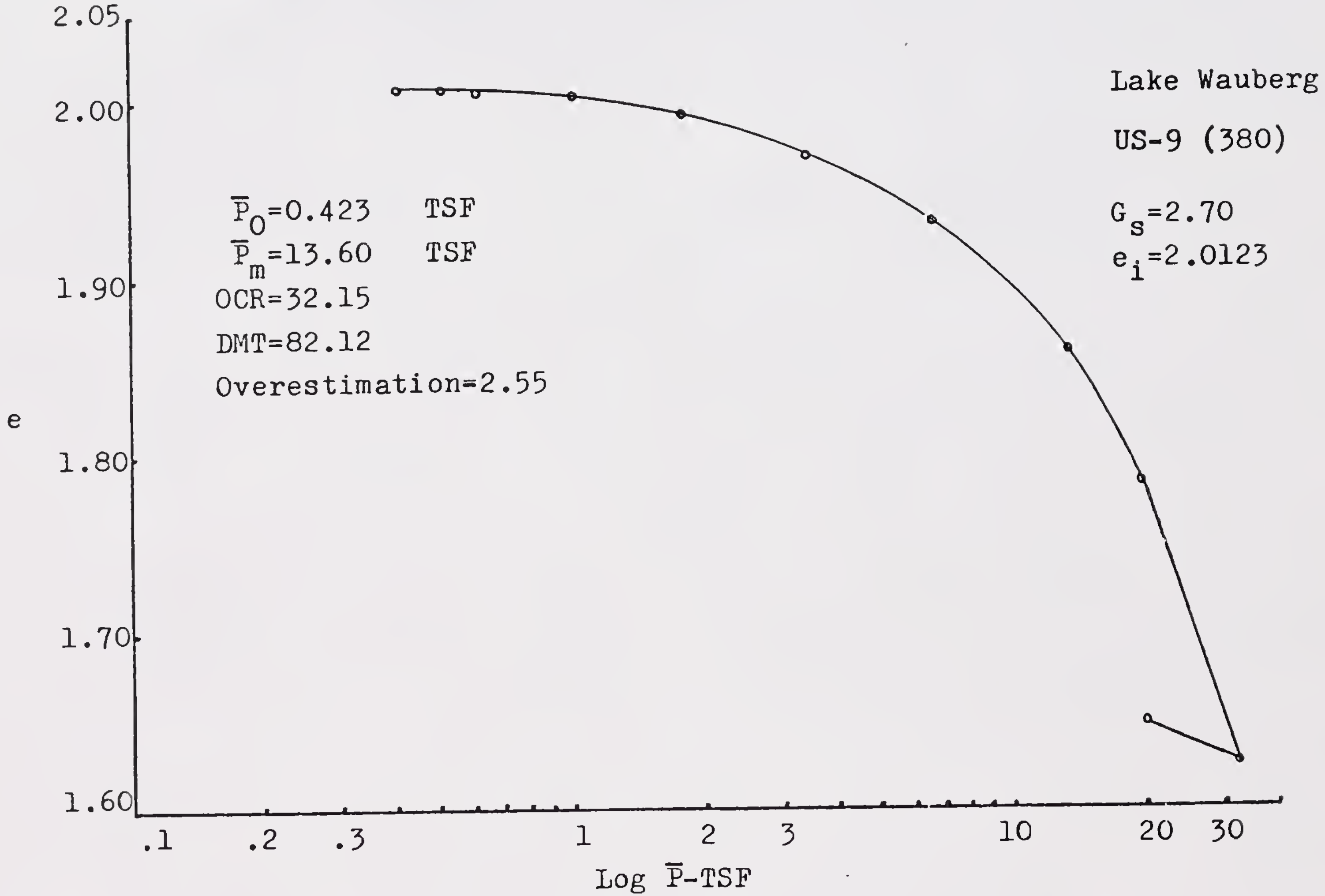


Figure D-14. e - log $\overline{P}$ plot from US-9 (380)

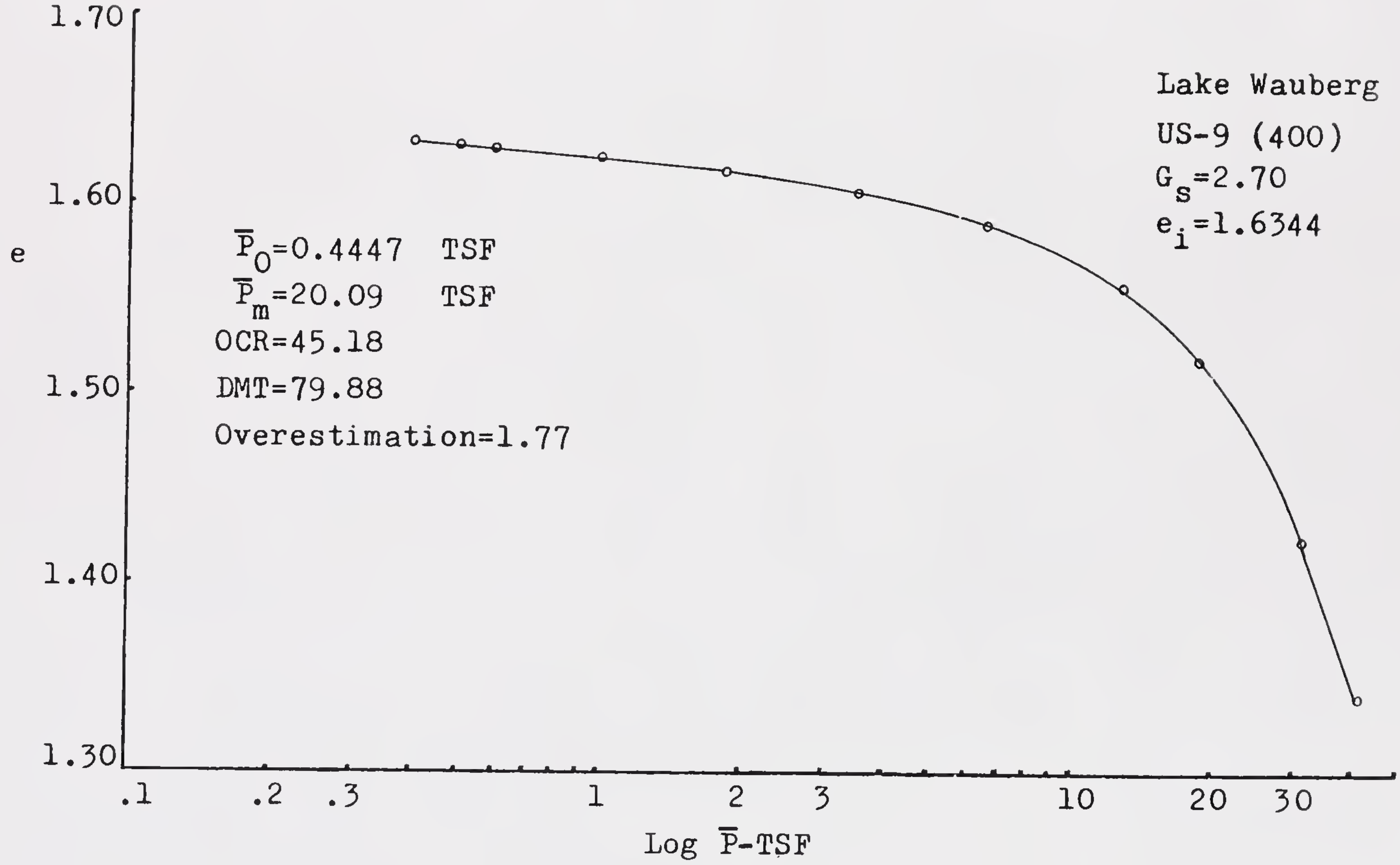


Figure D-15. e - log $\overline{P}$ plot from US-9 (400)

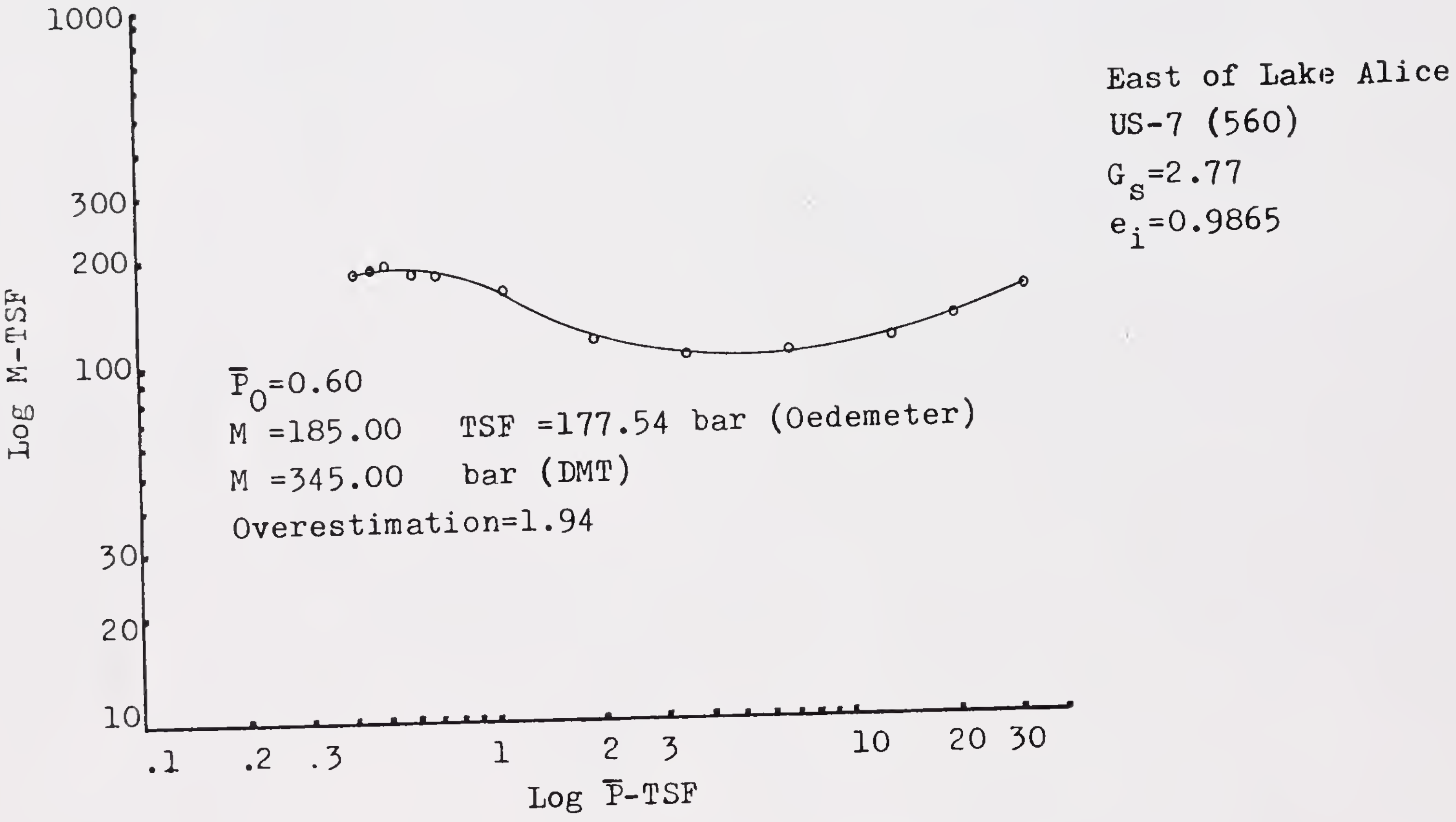


Figure D-16. log M - log $\overline{P}$ plot from US-7 (560)

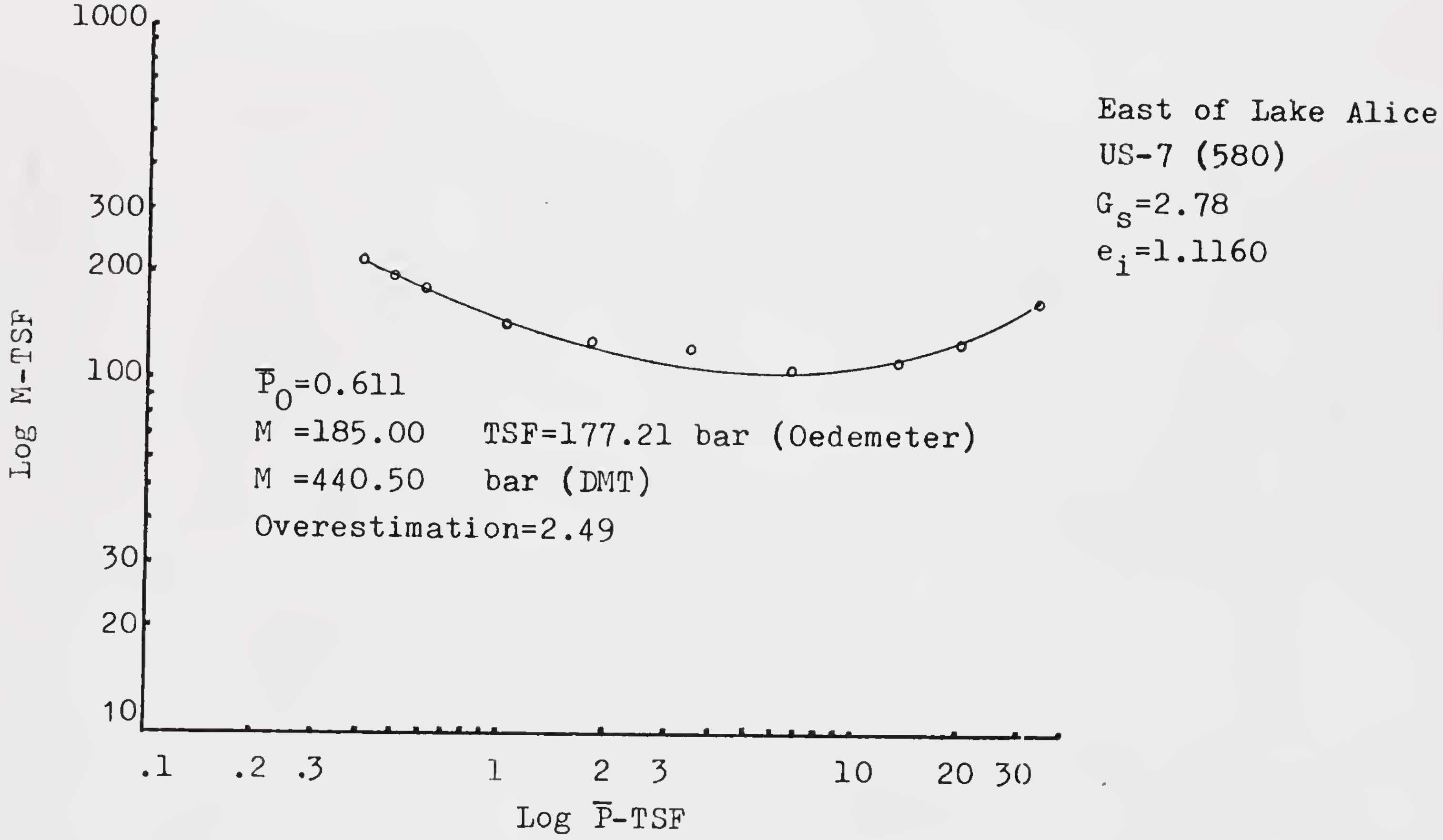


Figure D-17. log M - log $\overline{P}$ plot from US-7 (580)

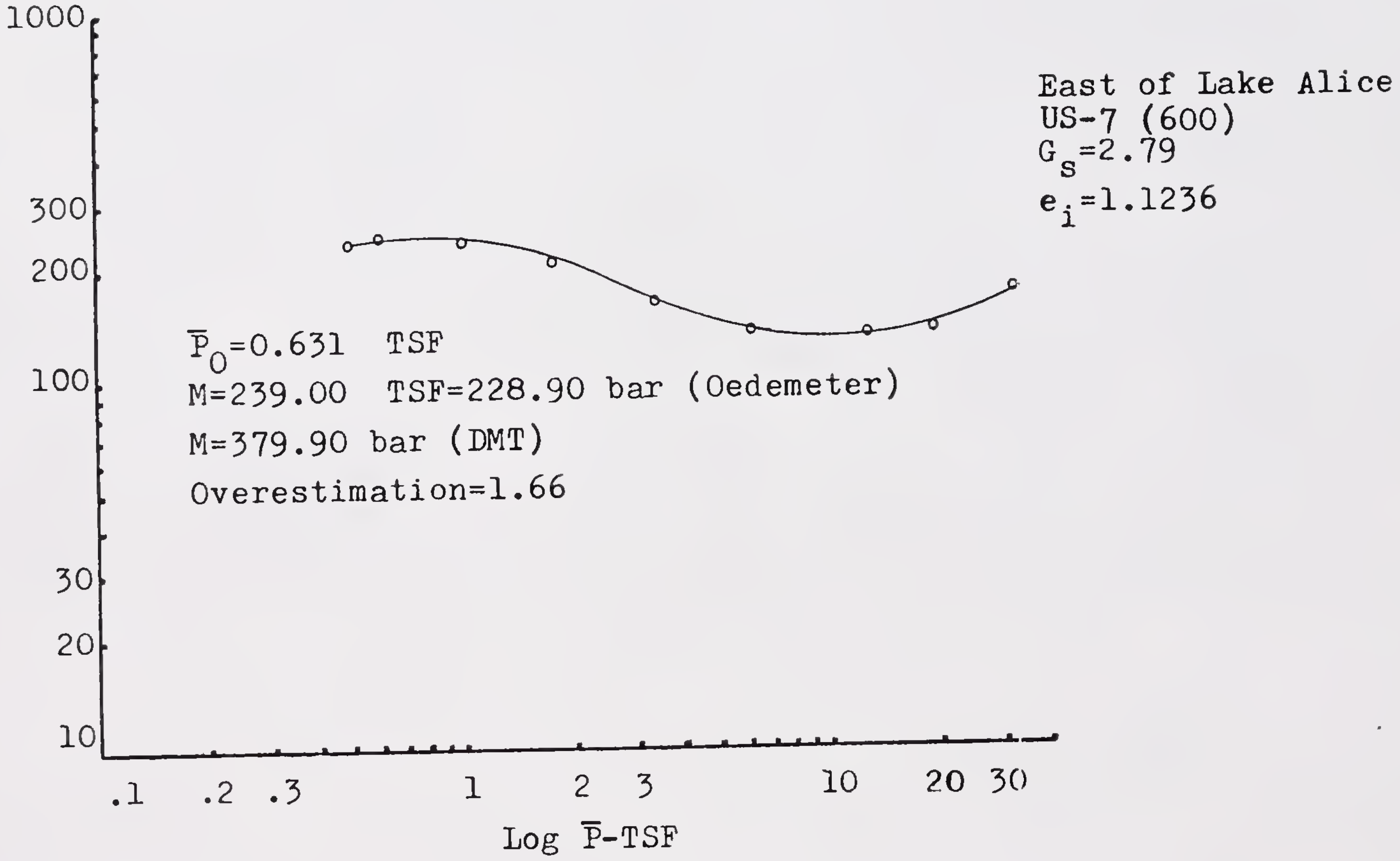


Figure D-18. log M - log $\overline{P}$ plot from US-7(600)

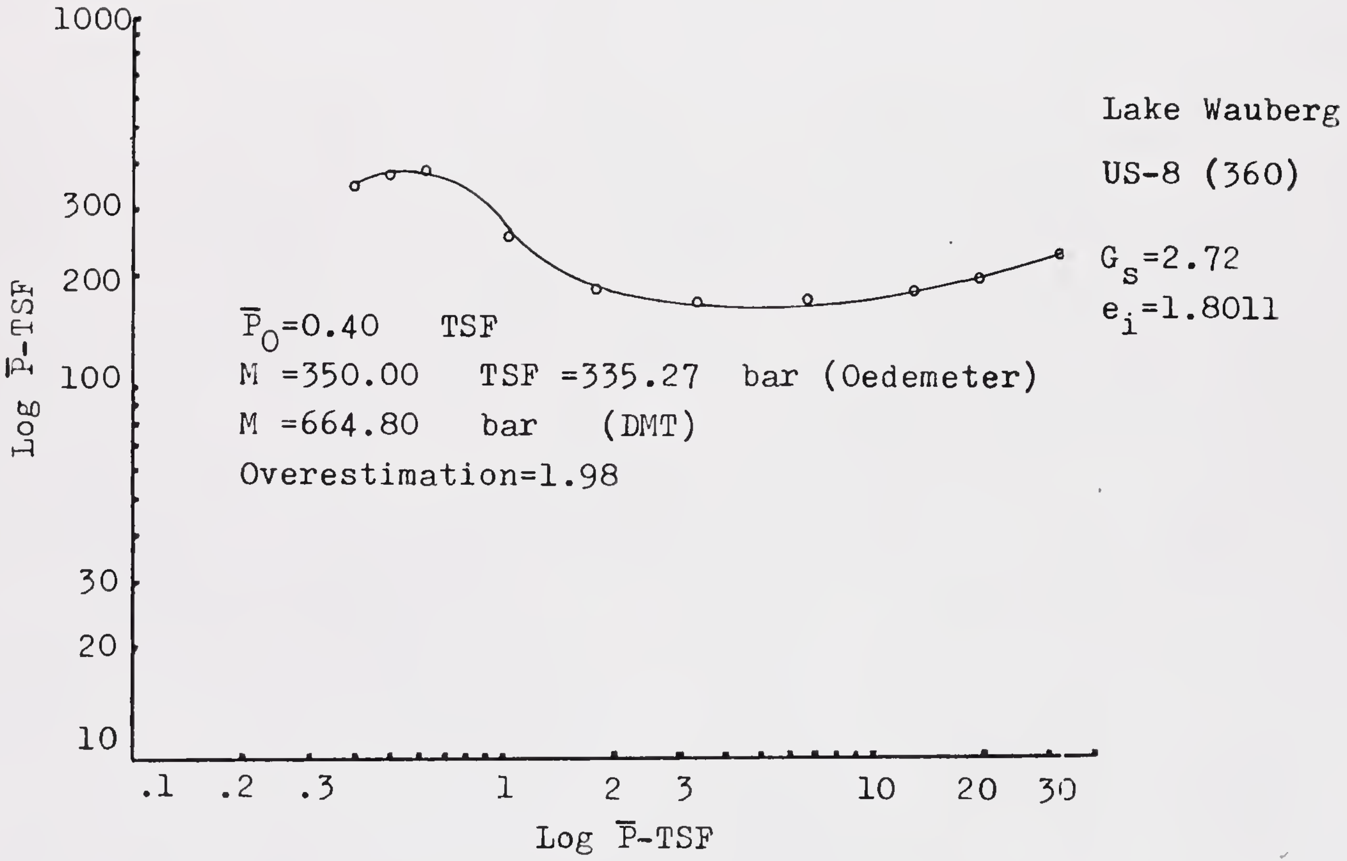


Figure D-19. log M - log $\overline{P}$ plot from US-8 (360)

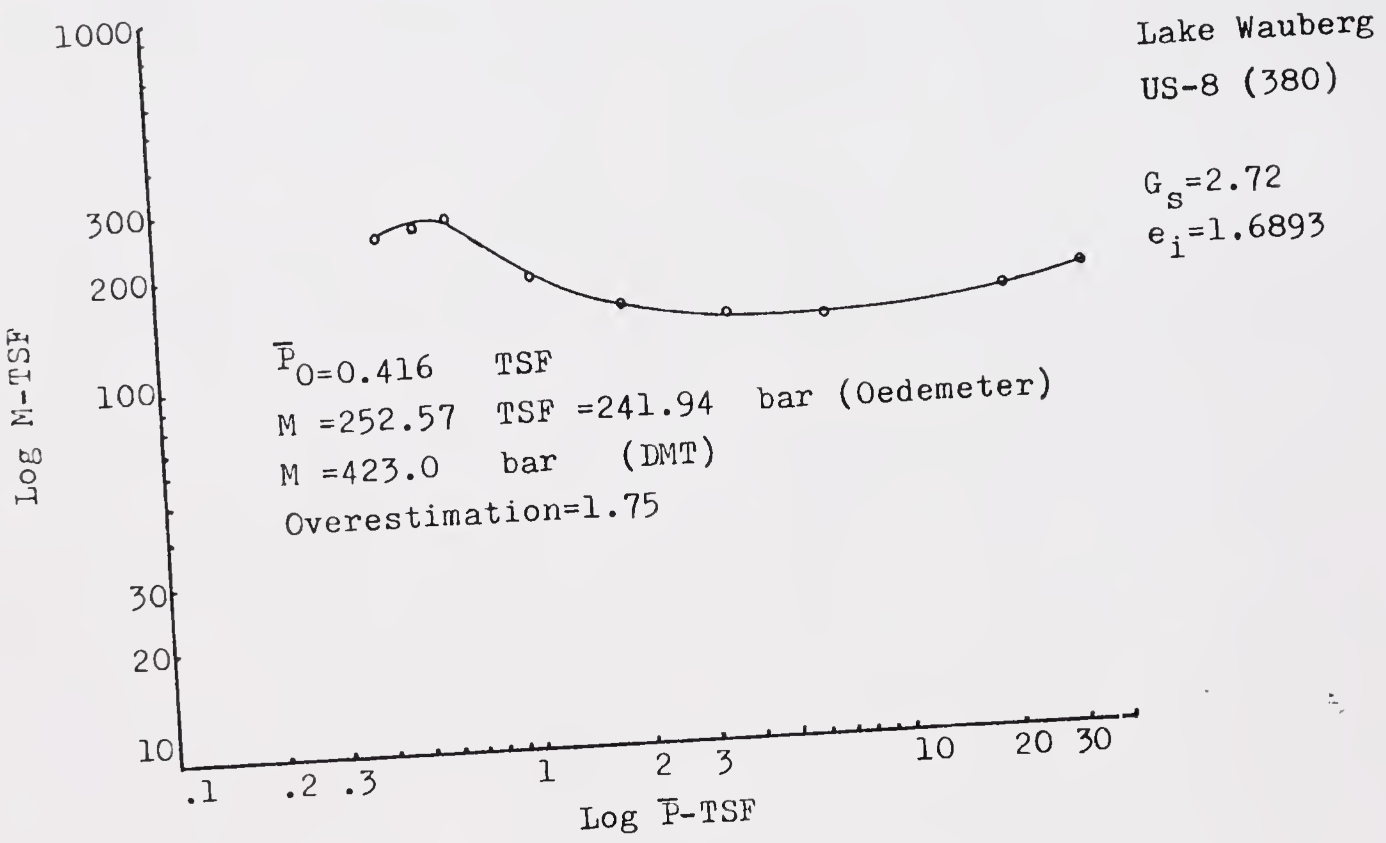


Figure D-20. log M - log $\overline{P}$ plot from US-8 (380)

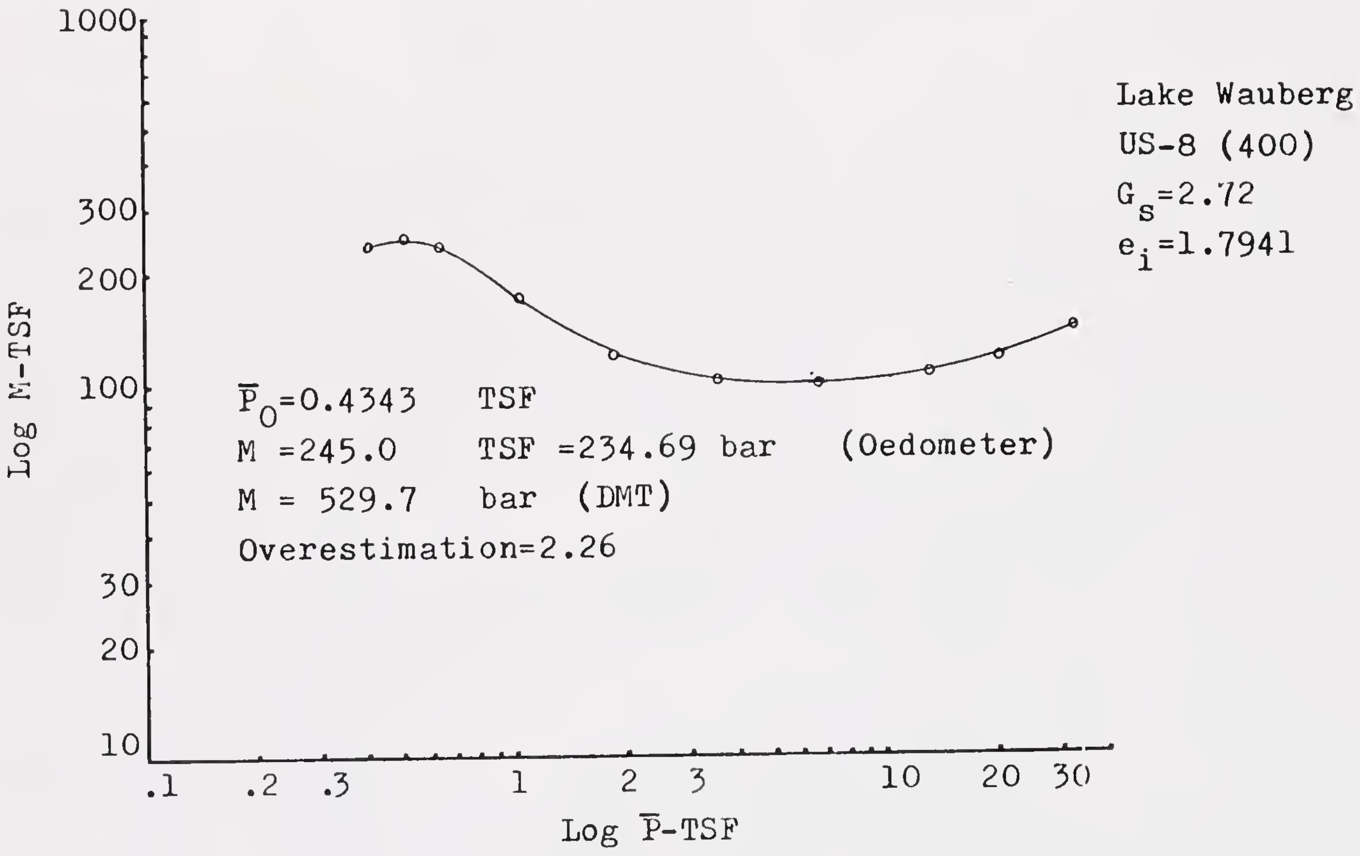


Figure D-21. log M - log $\overline{P}$ plot from US-8 (400)

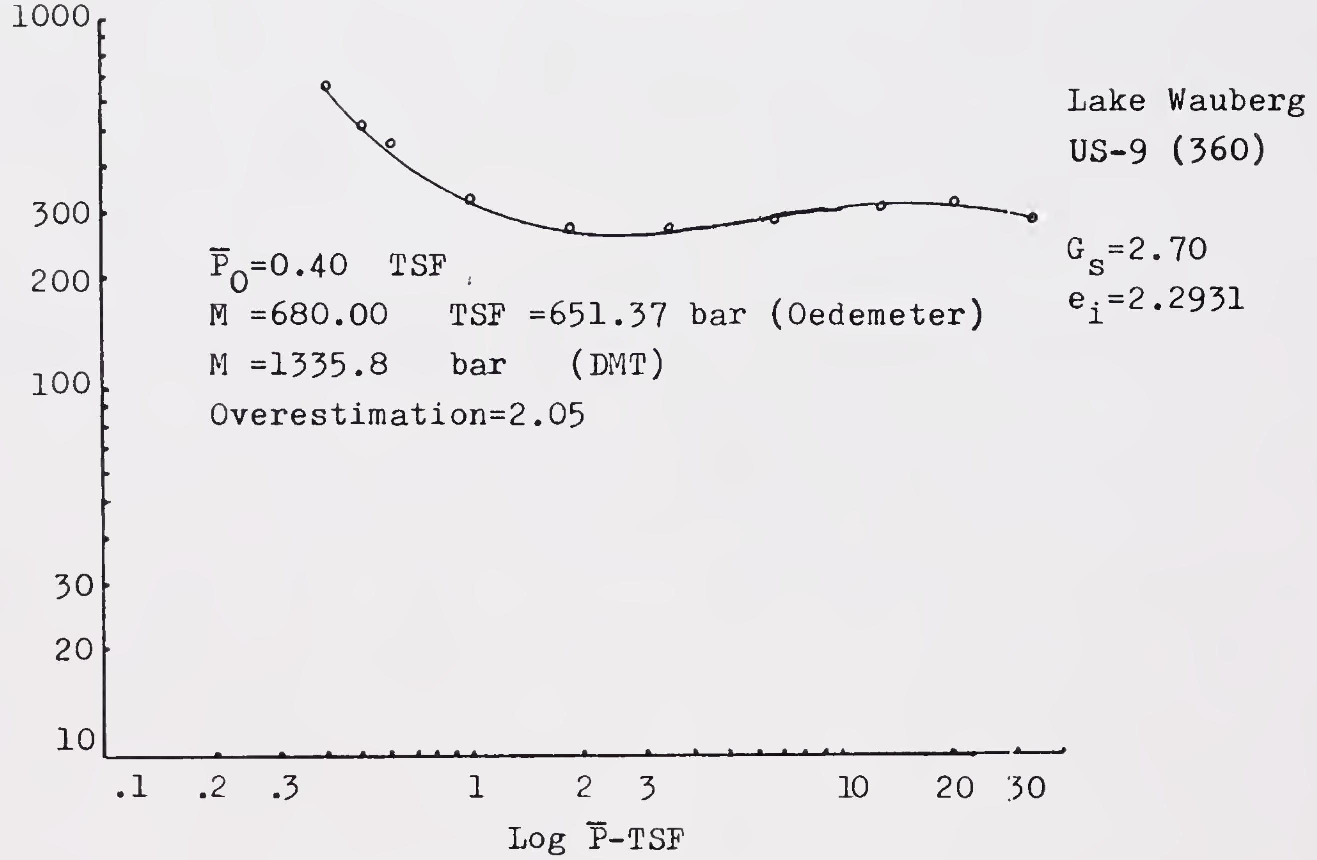


Figure D-22. log M - log $\overline{P}$ plot from US-9 (360)

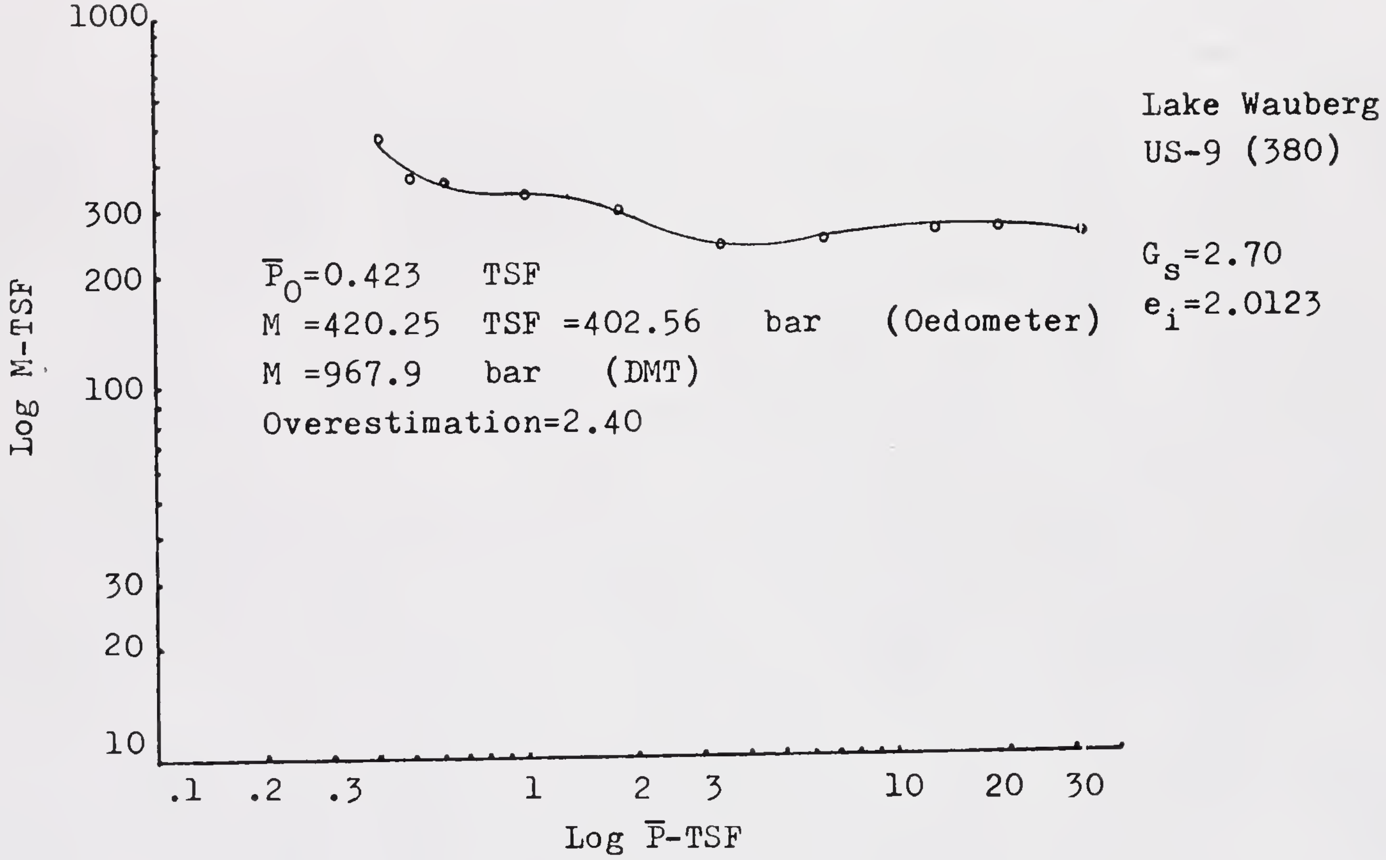


Figure D-23. log M - log $\overline{P}$ plot from US-9 (380)

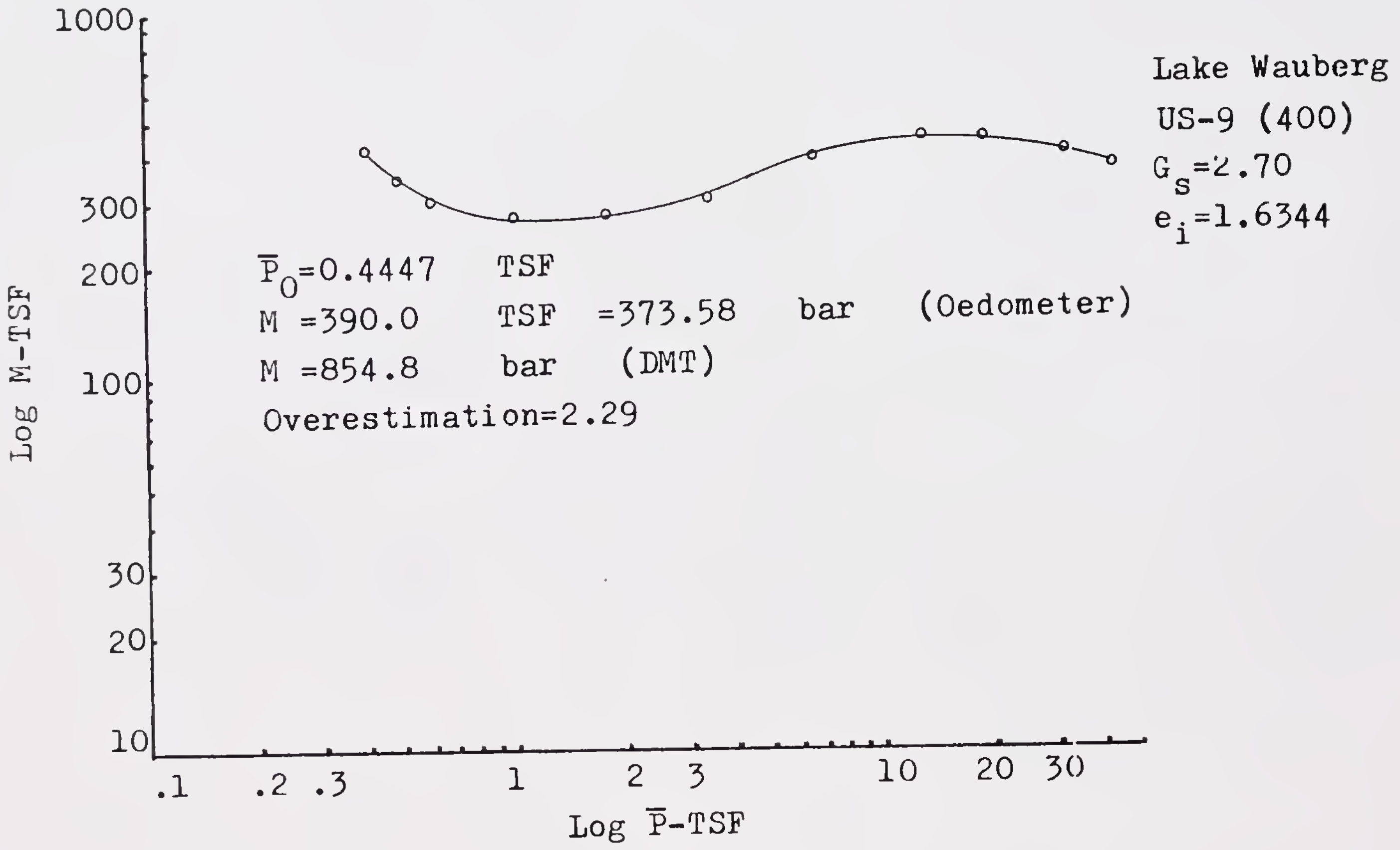


Figure D-24. log M - log $\overline{P}$ plot from US-9 (400)

APPENDIX E

TRIAXIAL TEST RESULTS

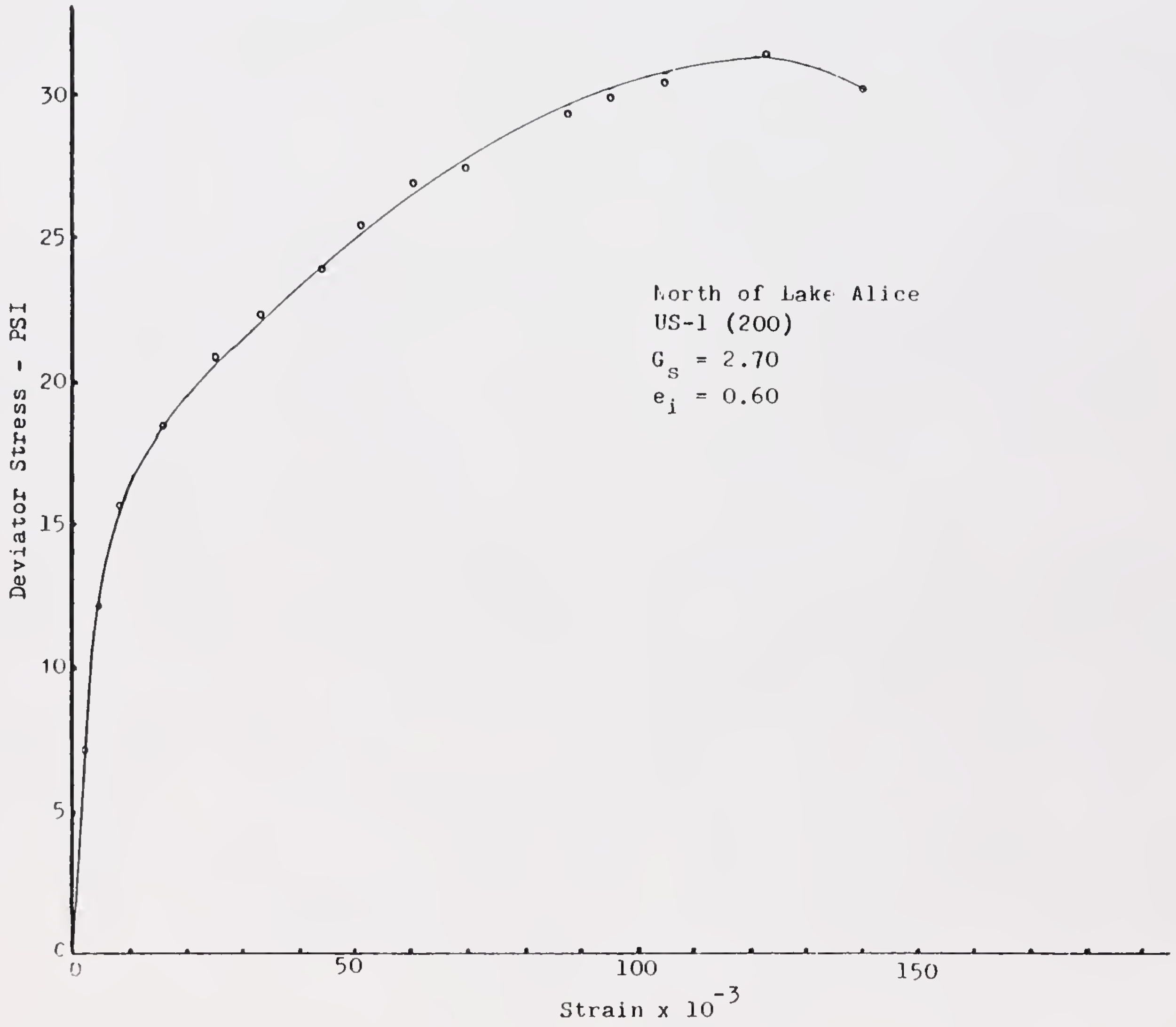


Figure E-1. Deviator Stress - Axial Strain plot from US-1 (200)

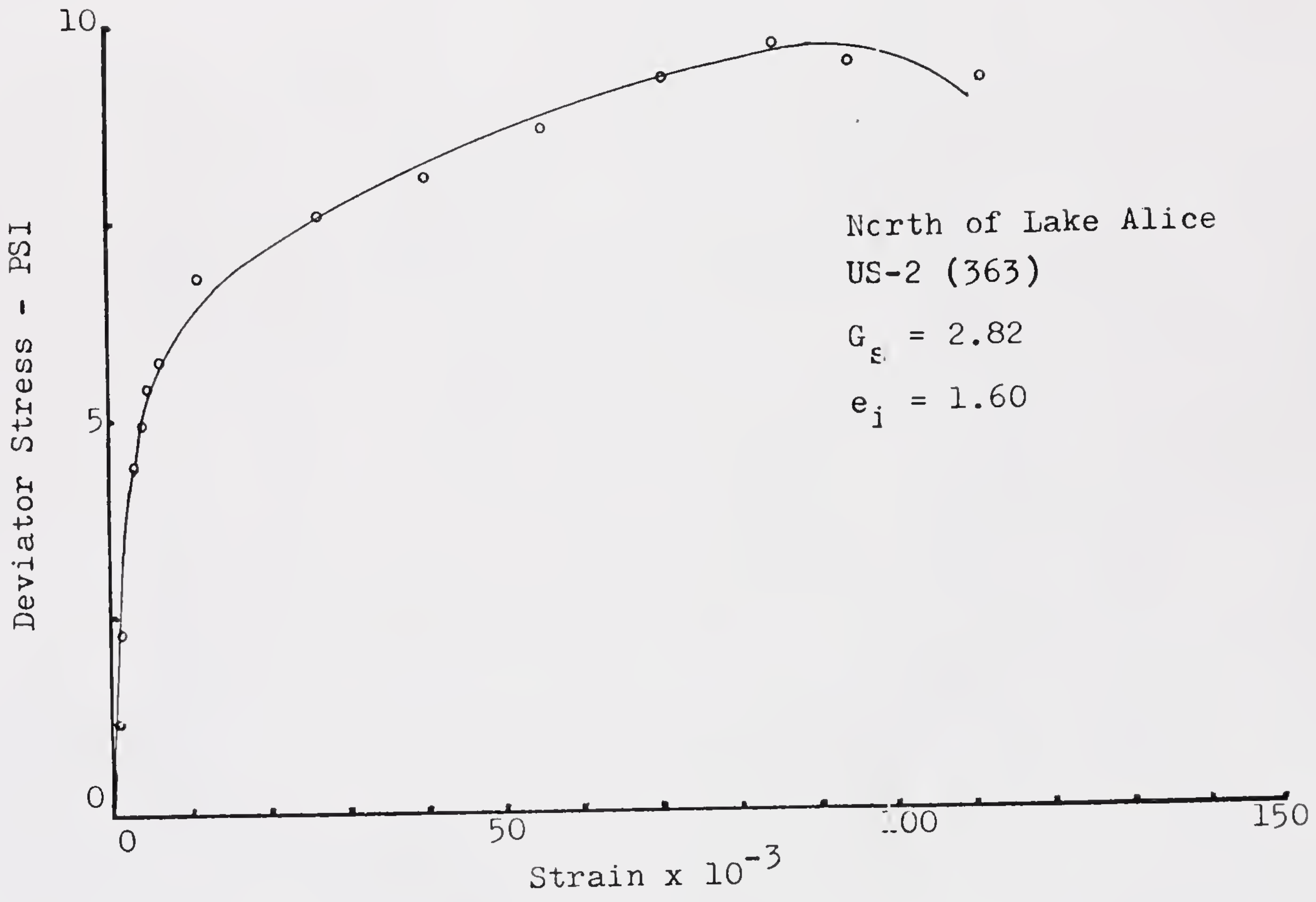


Figure E-2. Deviator Stress - Axial Strain plot from US-2 (363)

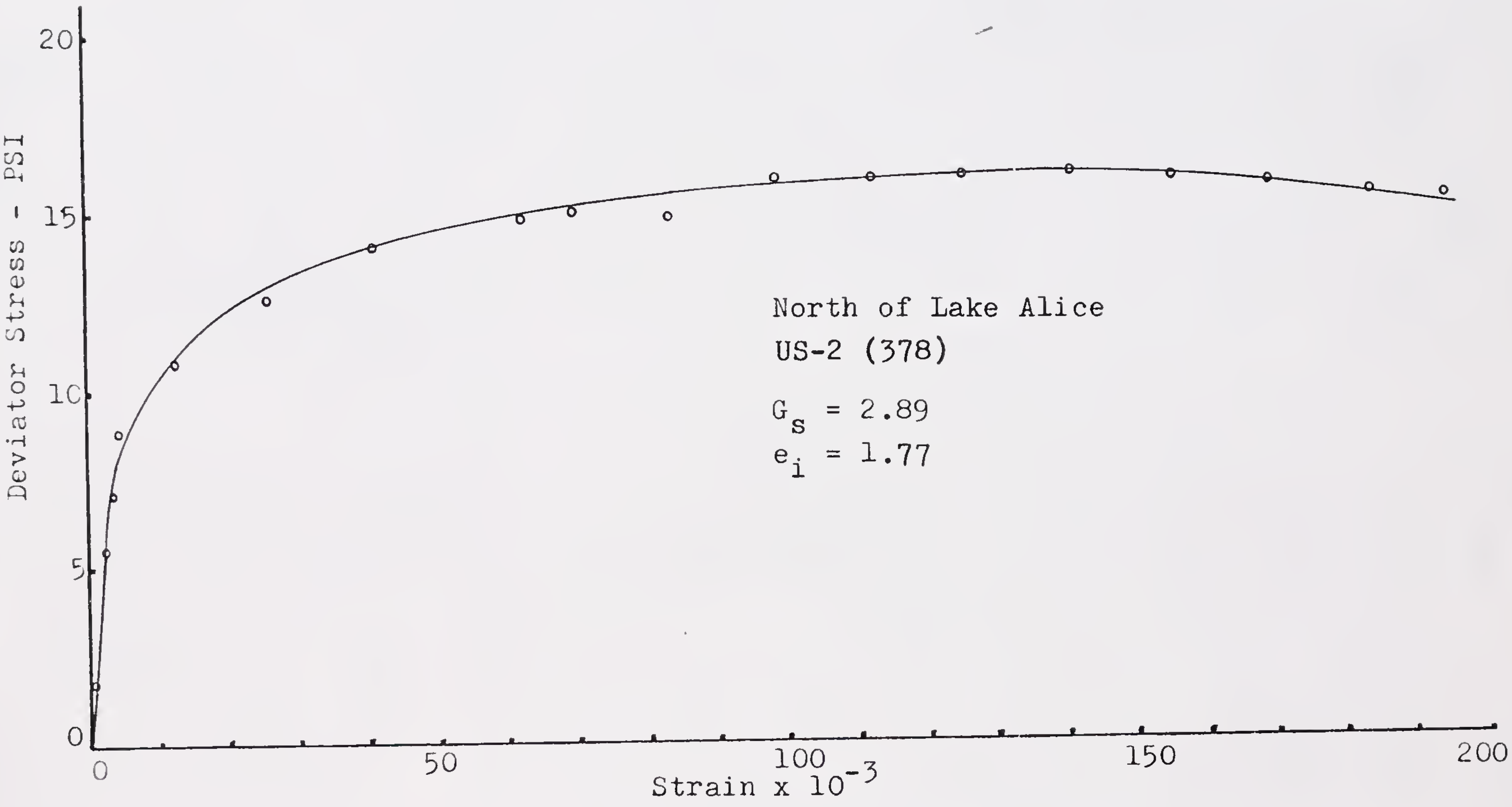


Figure E-3. Deviator Stress - Axial Strain plot from US-2 (378)

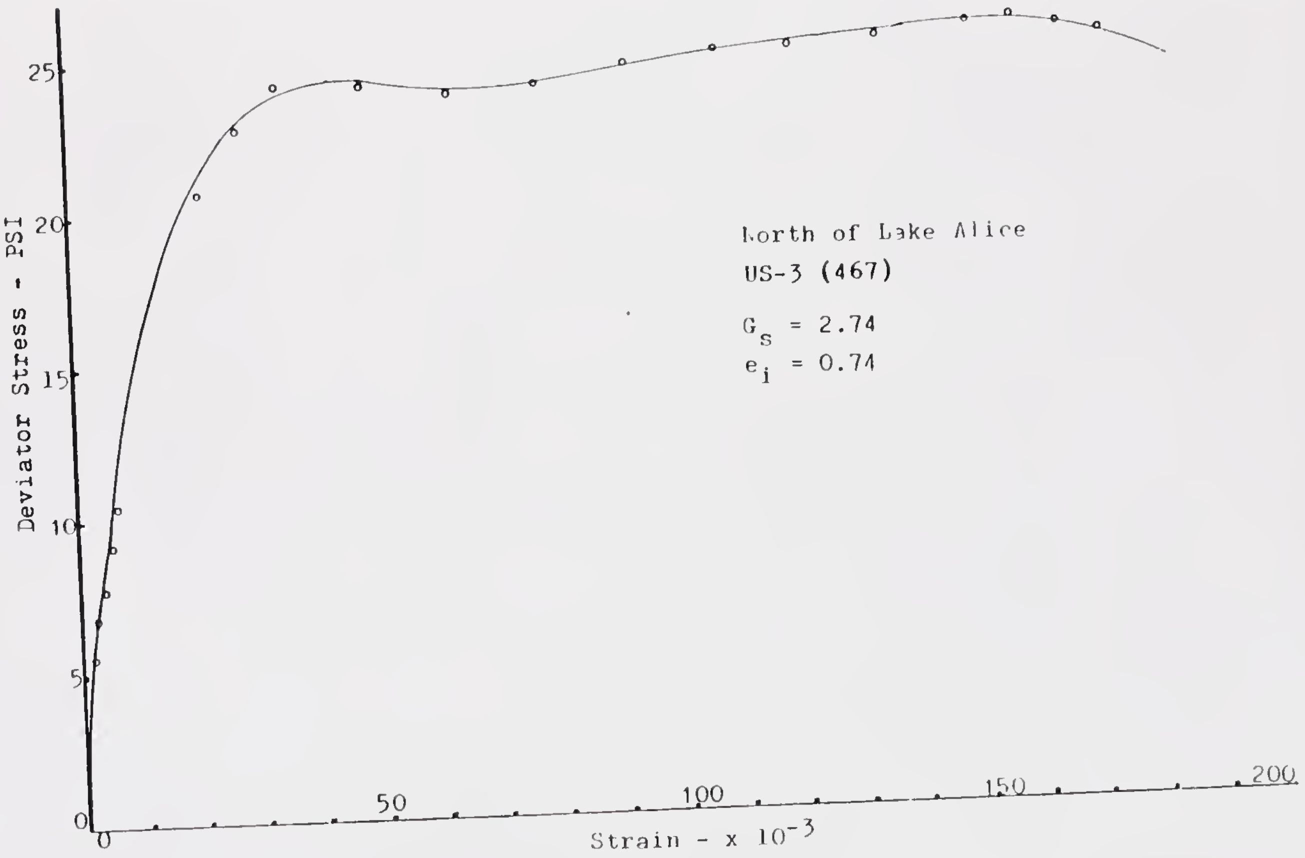


Figure E-4. Deviator Stress - Axial Strain plot from US-3 (467)

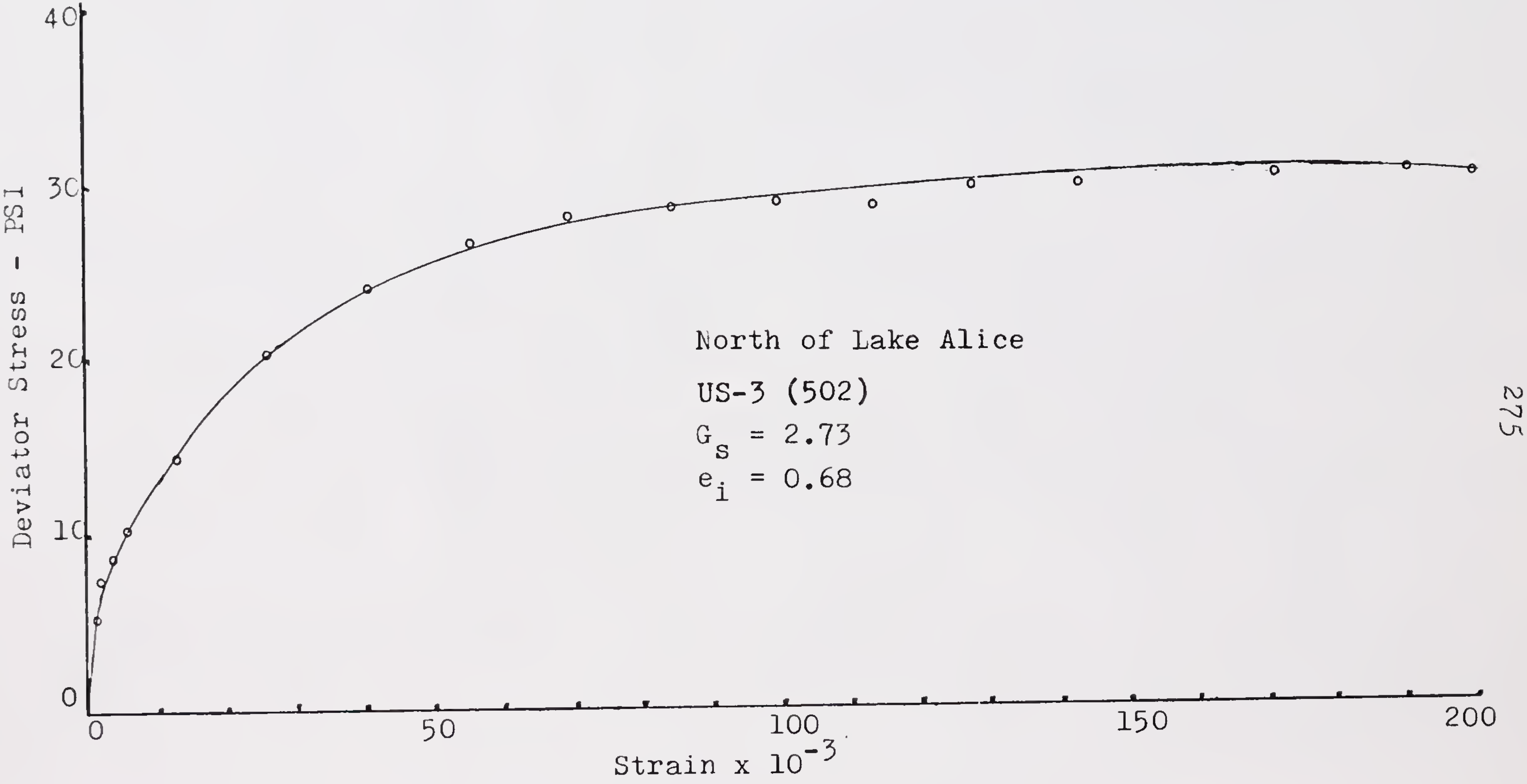


Figure E-5. Deviator Stress - Axial Strain plot from US-3 (502)

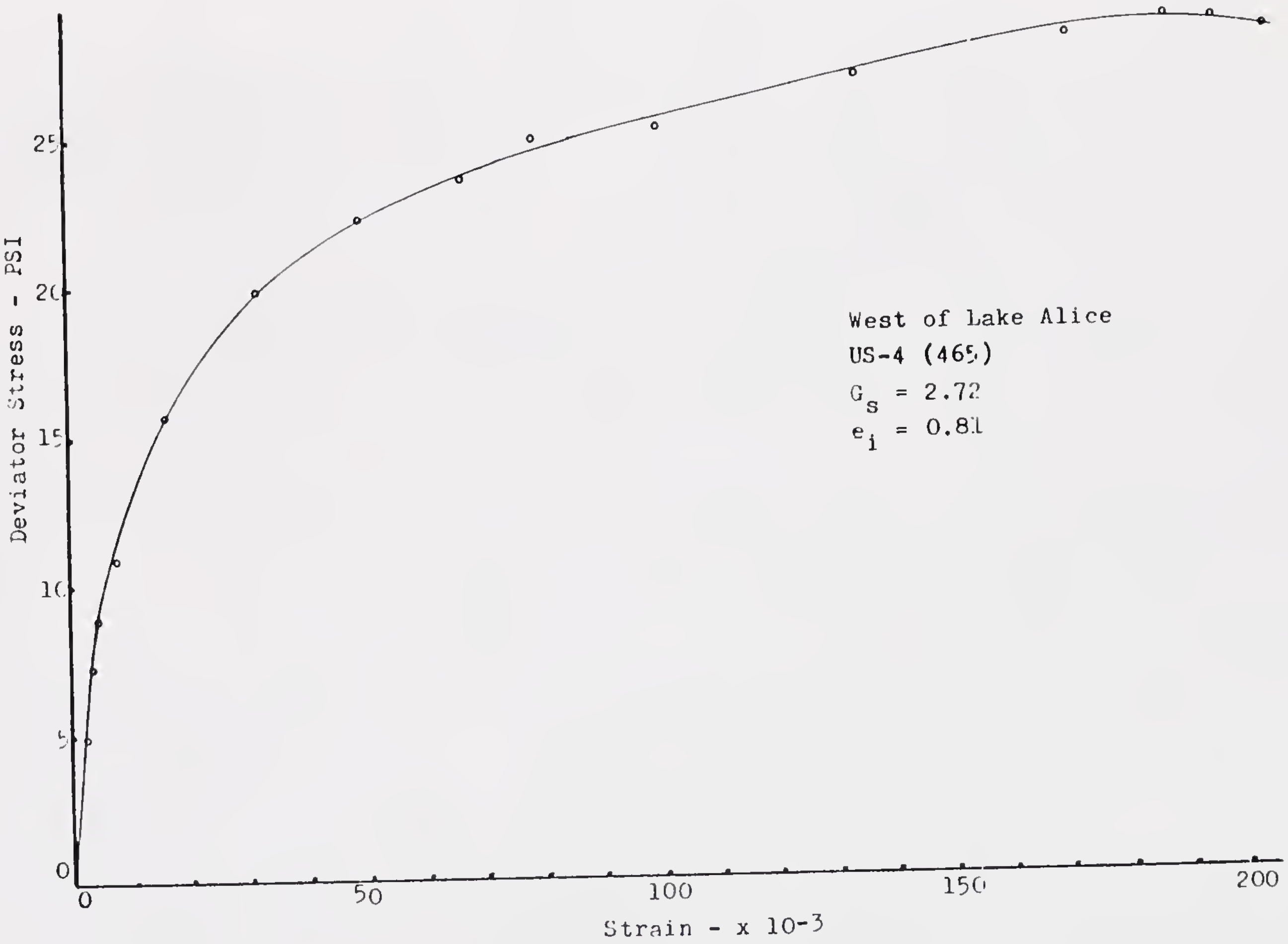


Figure E-6. Deviator Stress - Axial Strain plot from US-4 (465)

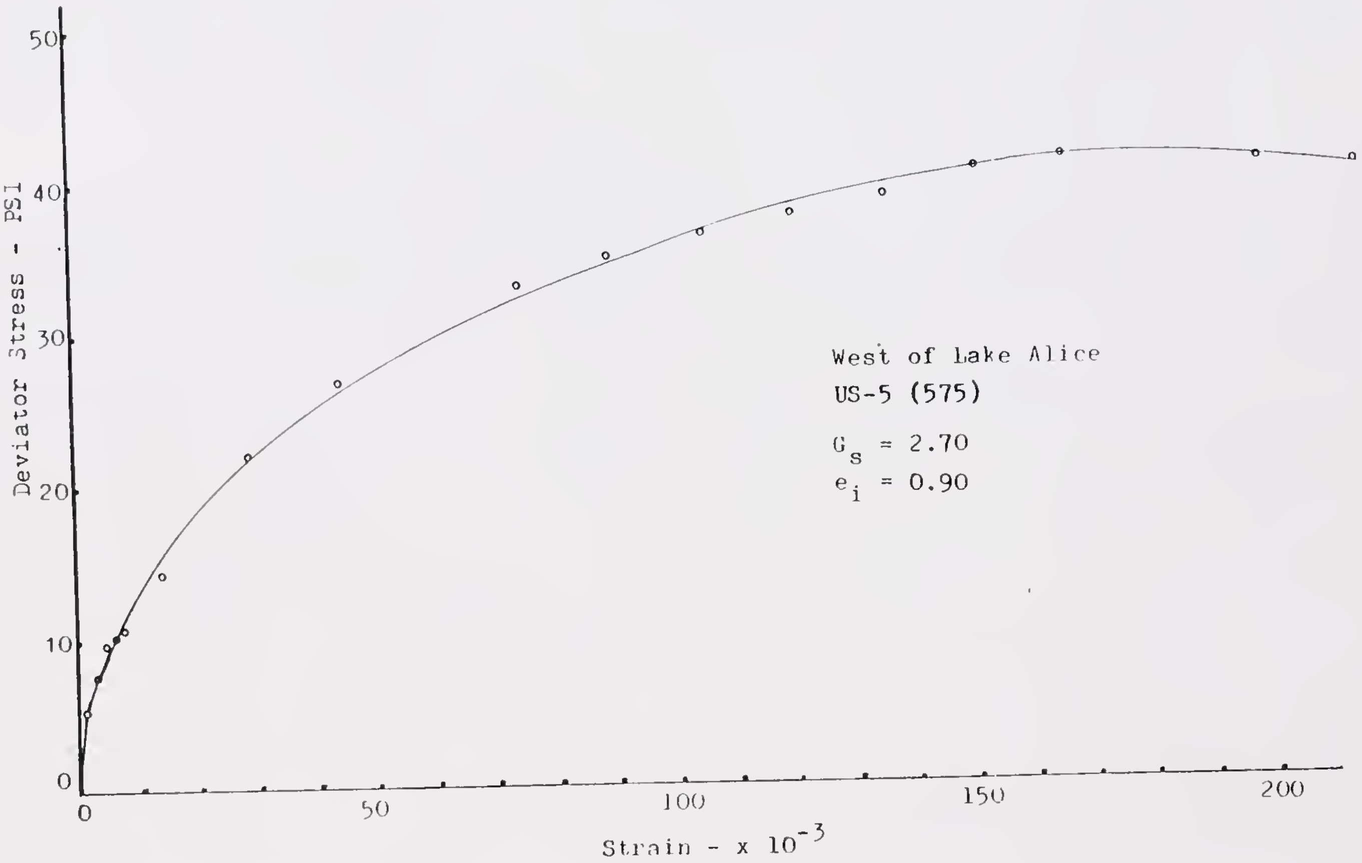


Figure E-7. Deviator Stress - Axial Strain plot from US-5 (575)

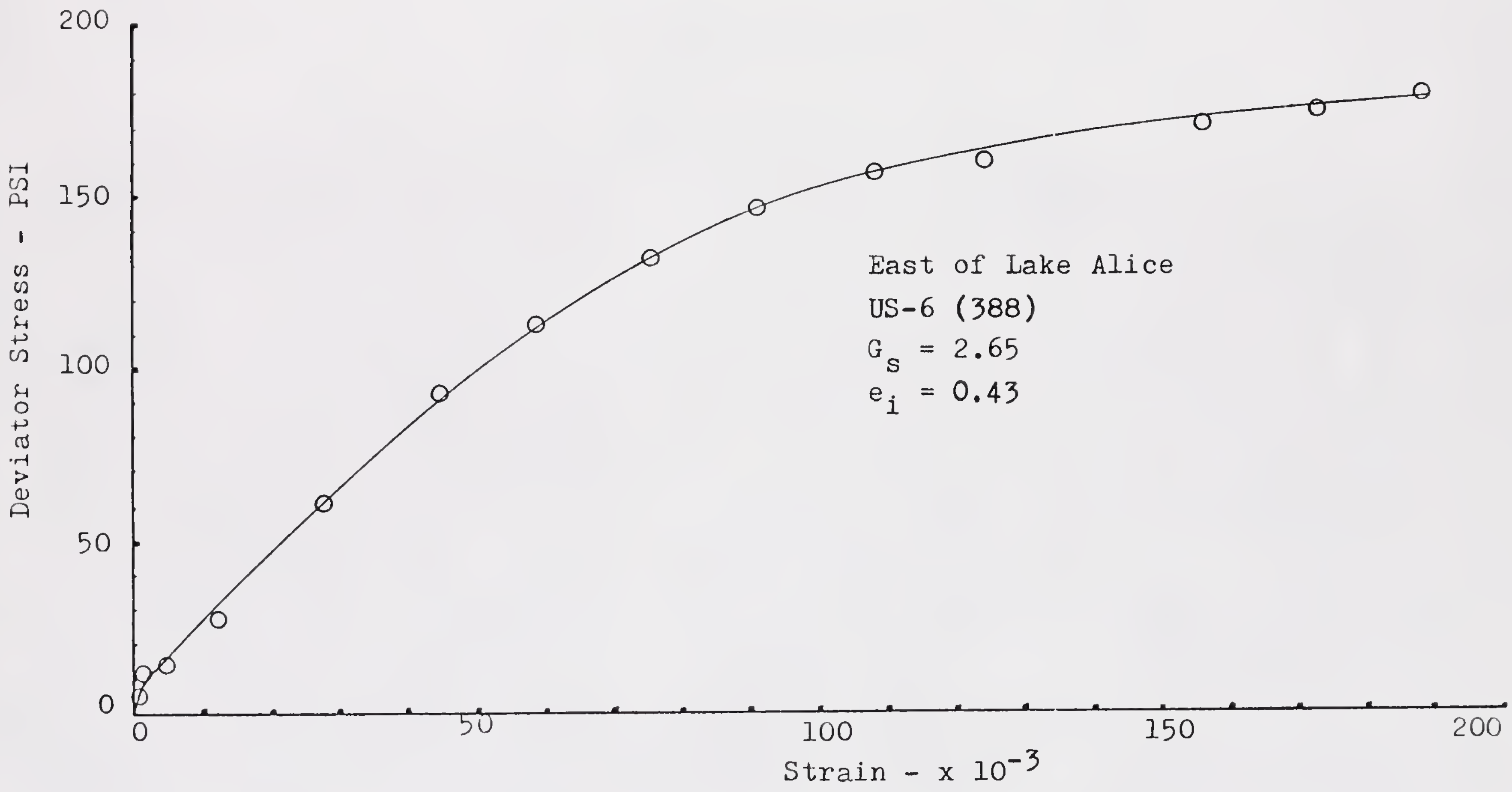


Figure E-8. Deviator Stress - Axial Strain plot from US-6 (388)

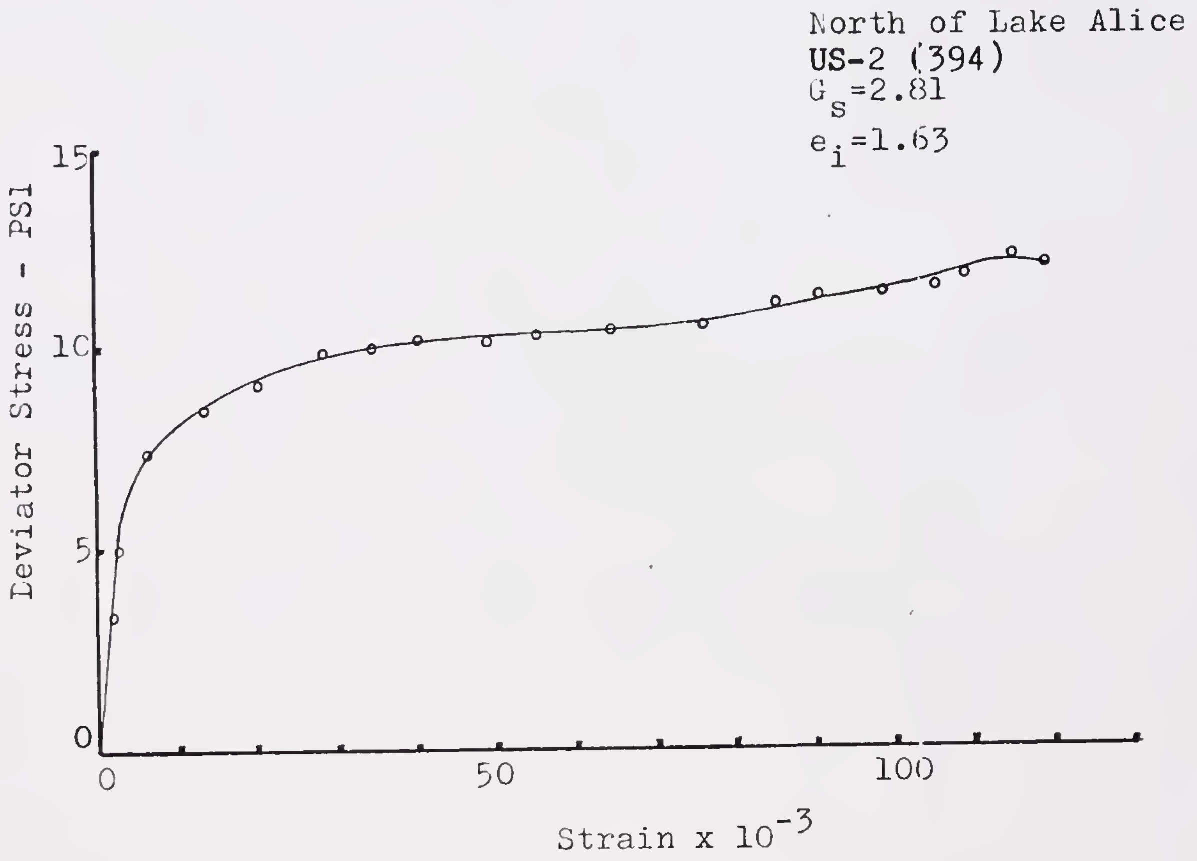


Figure E-9. Deviator Stress - Axial Strain plot from US-2 (394)

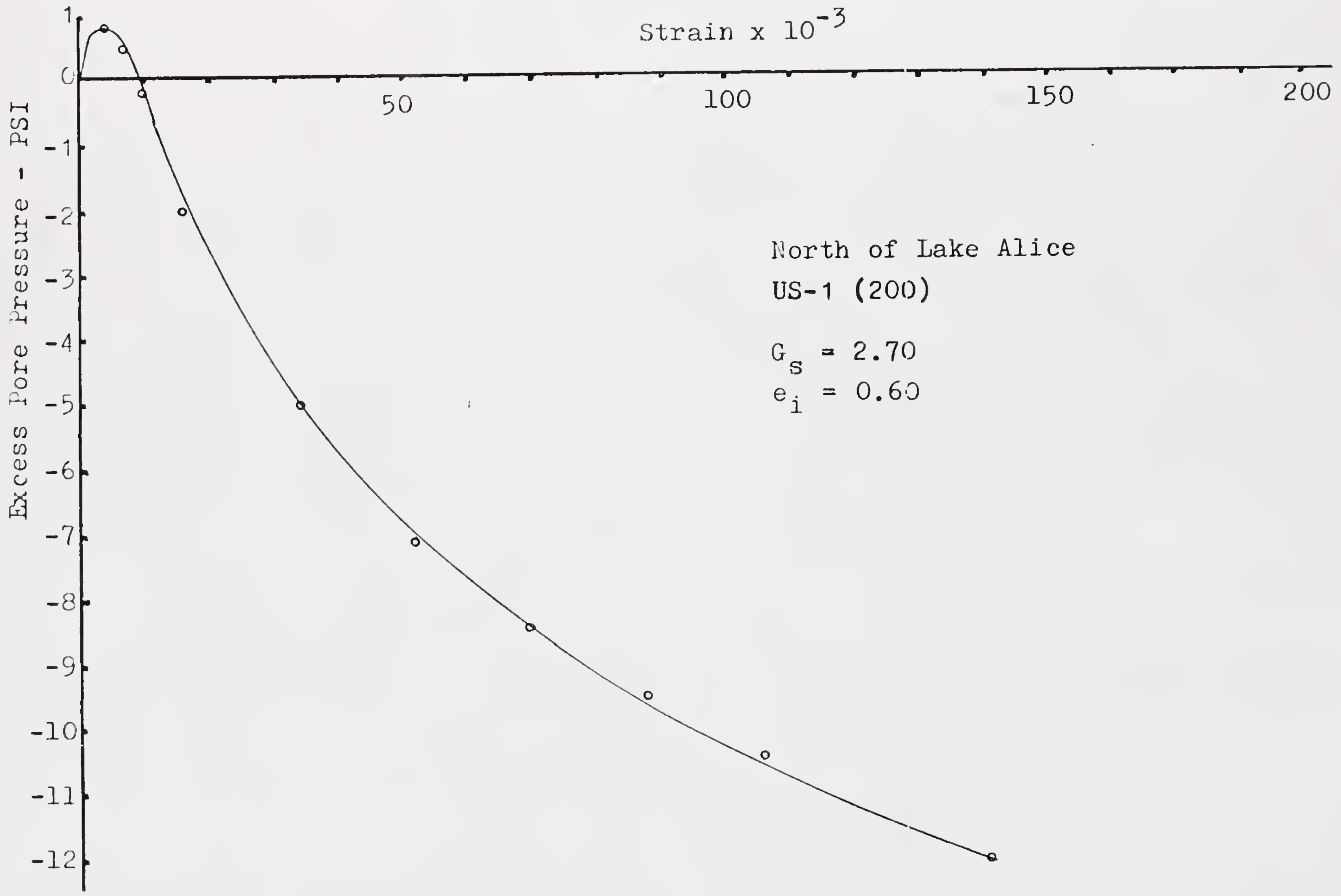


Figure E-10. Excess Pore Pressure - Axial Strain plot from US-1 (200)

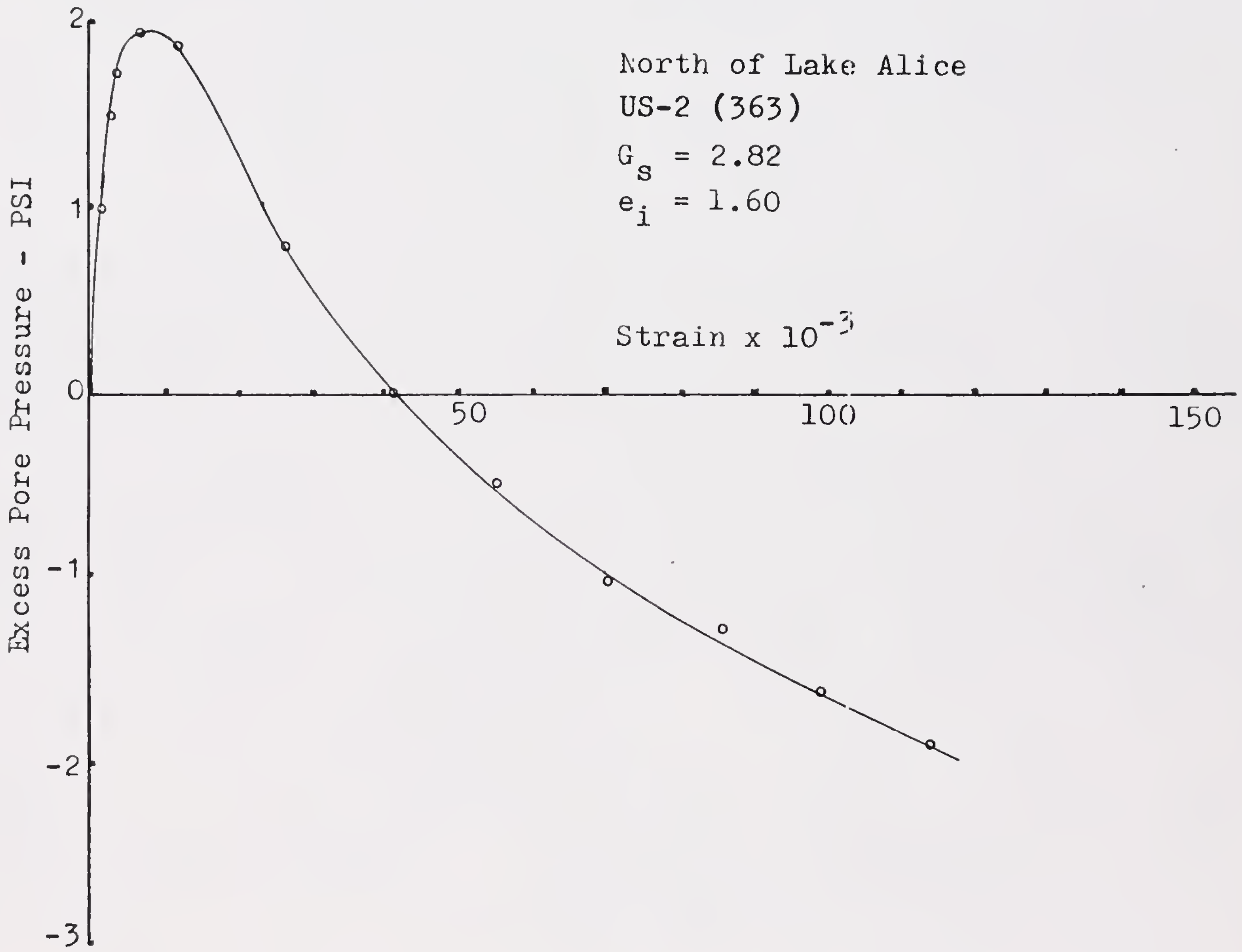


Figure E-11. Excess Pore Pressure - Axial Strain plot from US-2 (363)

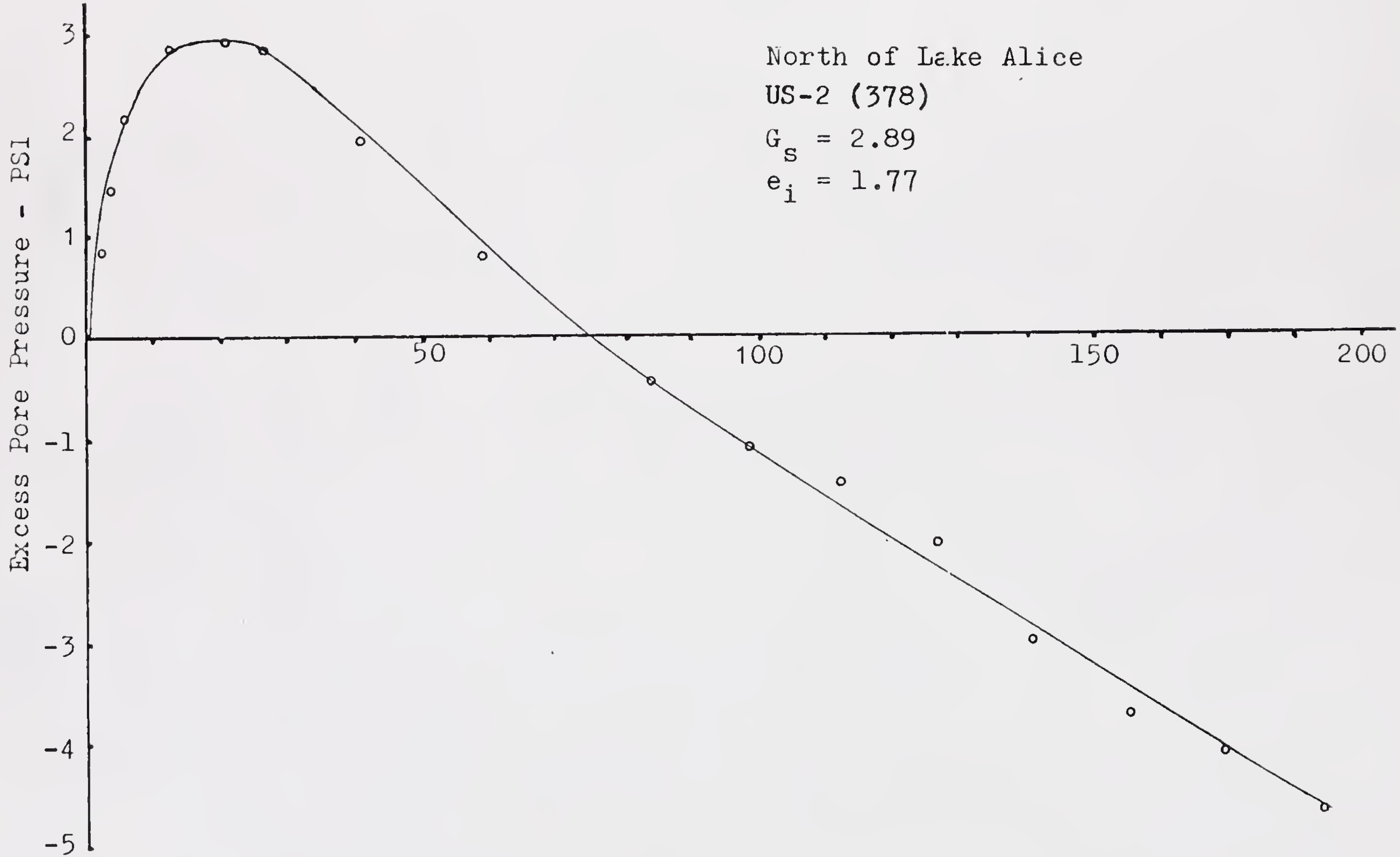


Figure E-12. Excess Pore Pressure - Axial Strain plot from US-2 (378)

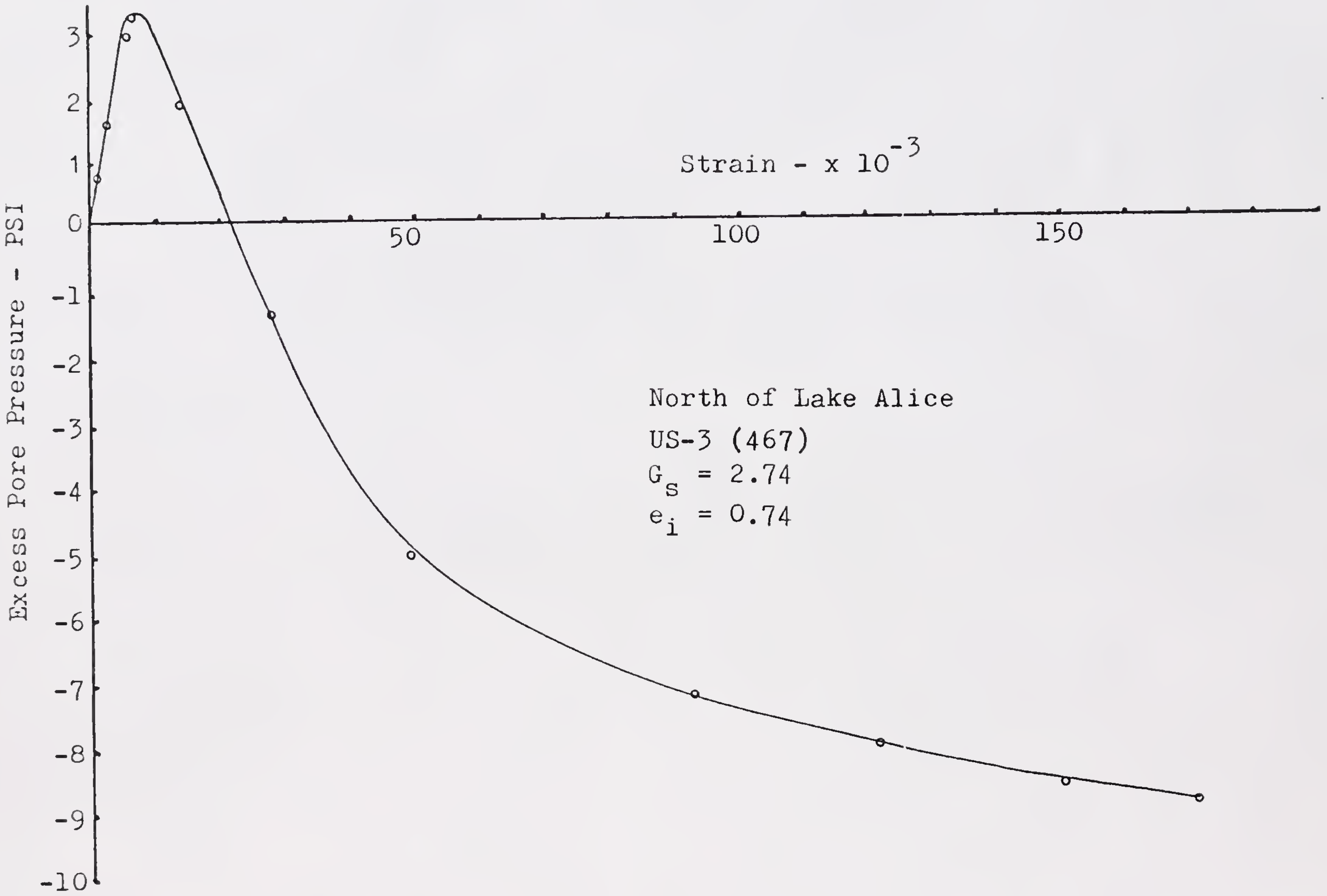


Figure E-13. Excess Pore Pressure - Axial Strain plot from US-3 (467)

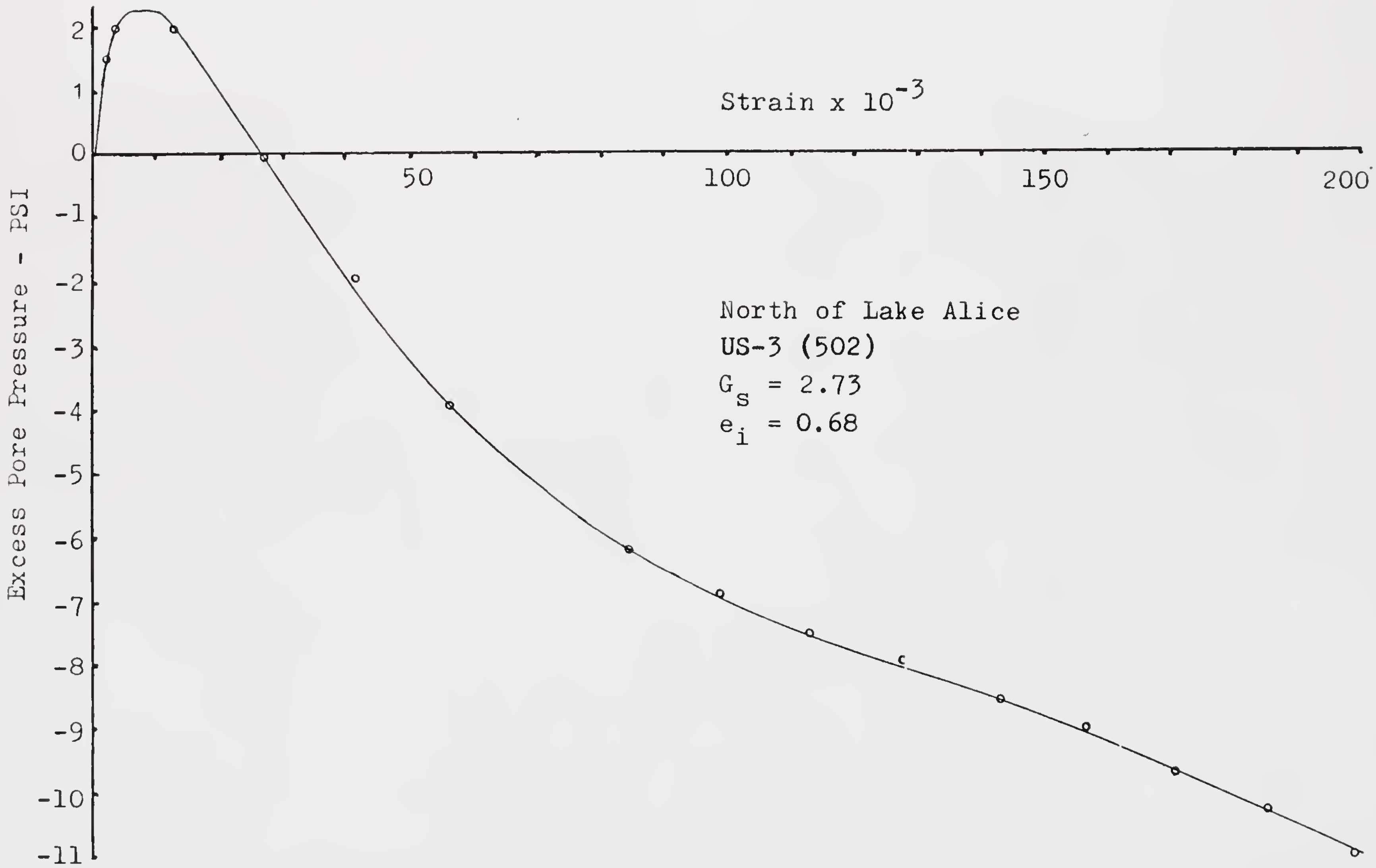


Figure E-14. Excess Pore Pressure - Axial Strain plot from US-3 (502)

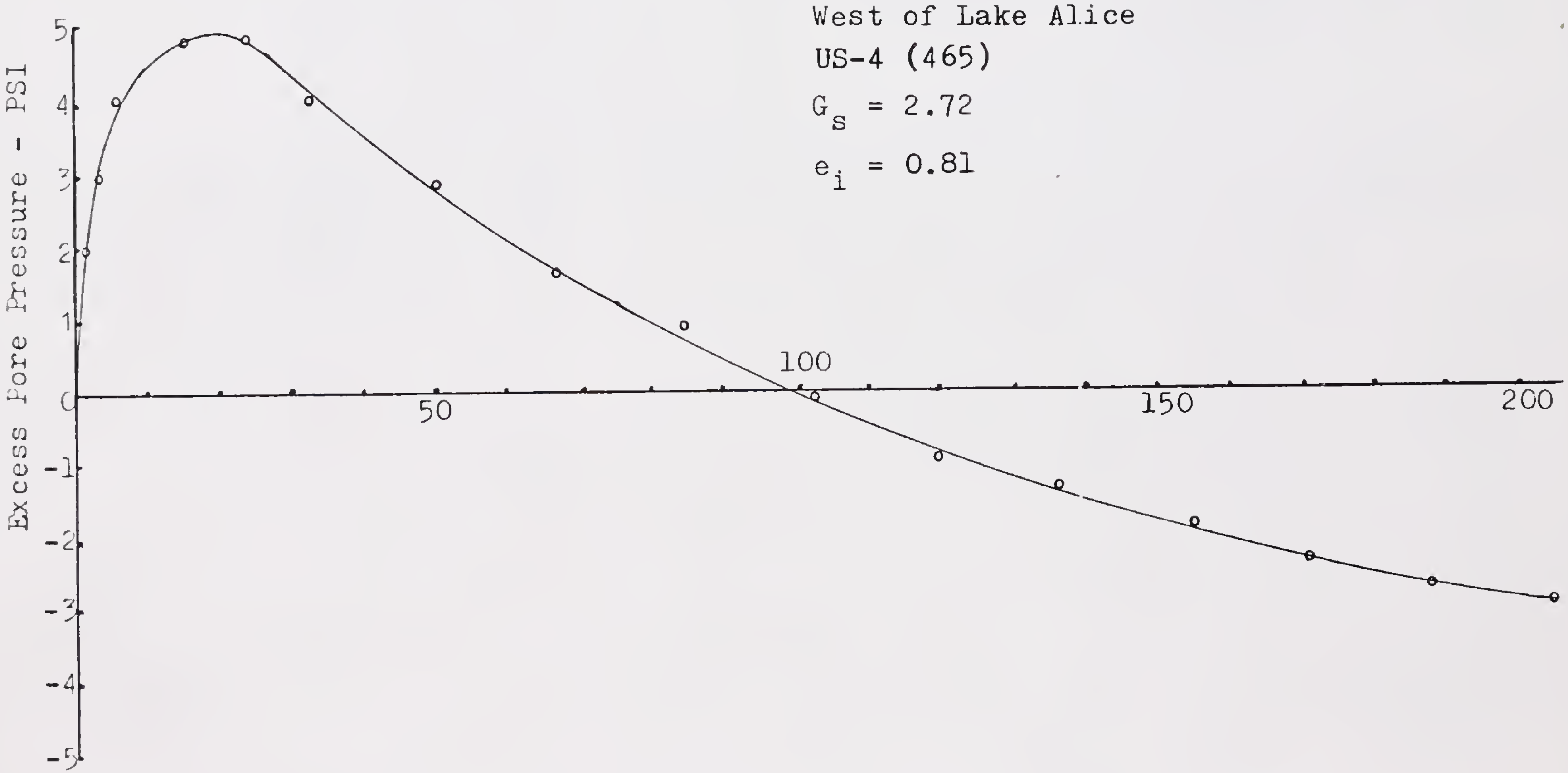


Figure E-15. Excess Pore Pressure - Axial Strain plot from US-4 (465)

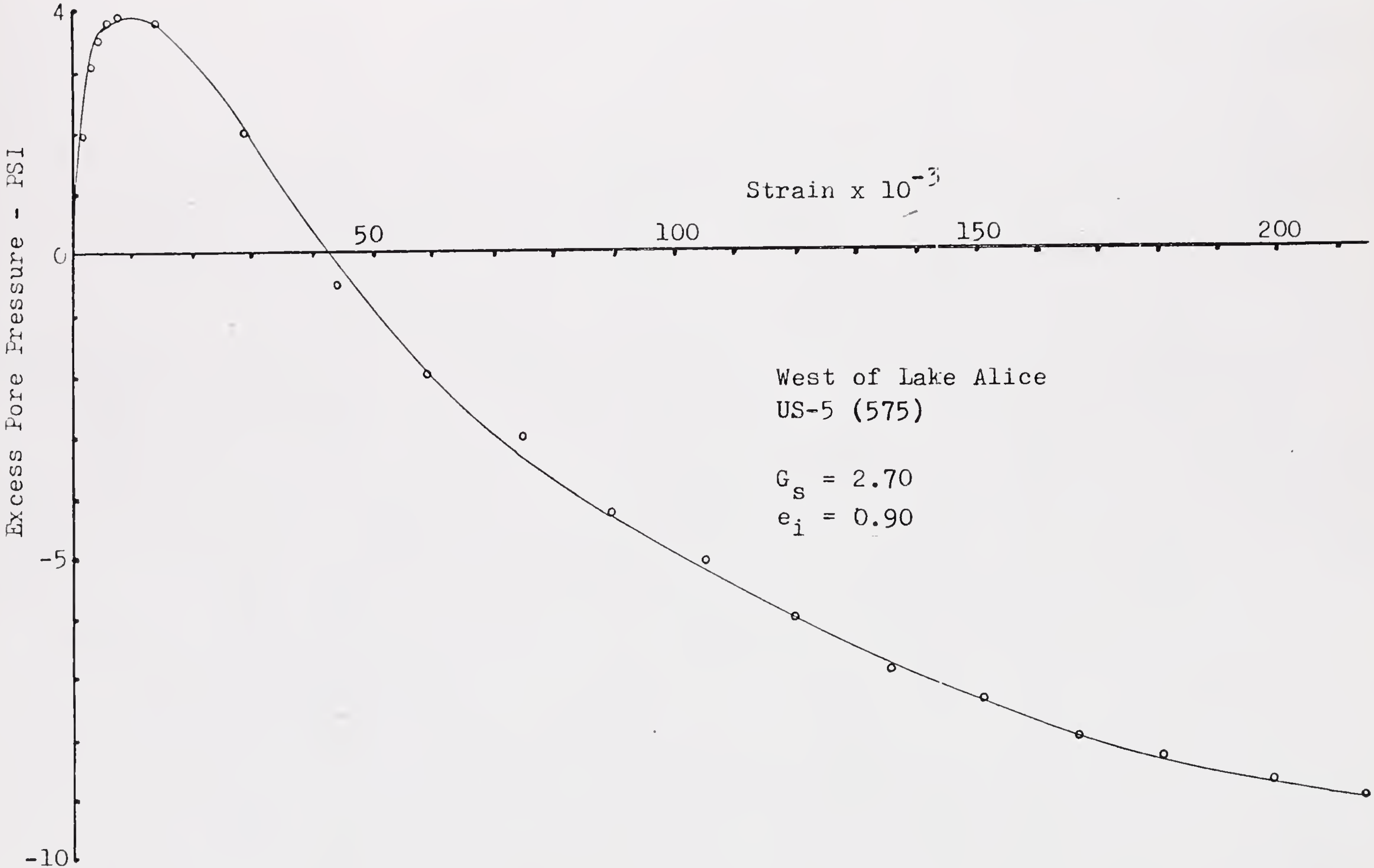


Figure E-16. Excess Pore Pressure - Axial Strain plot from US-5 (575)

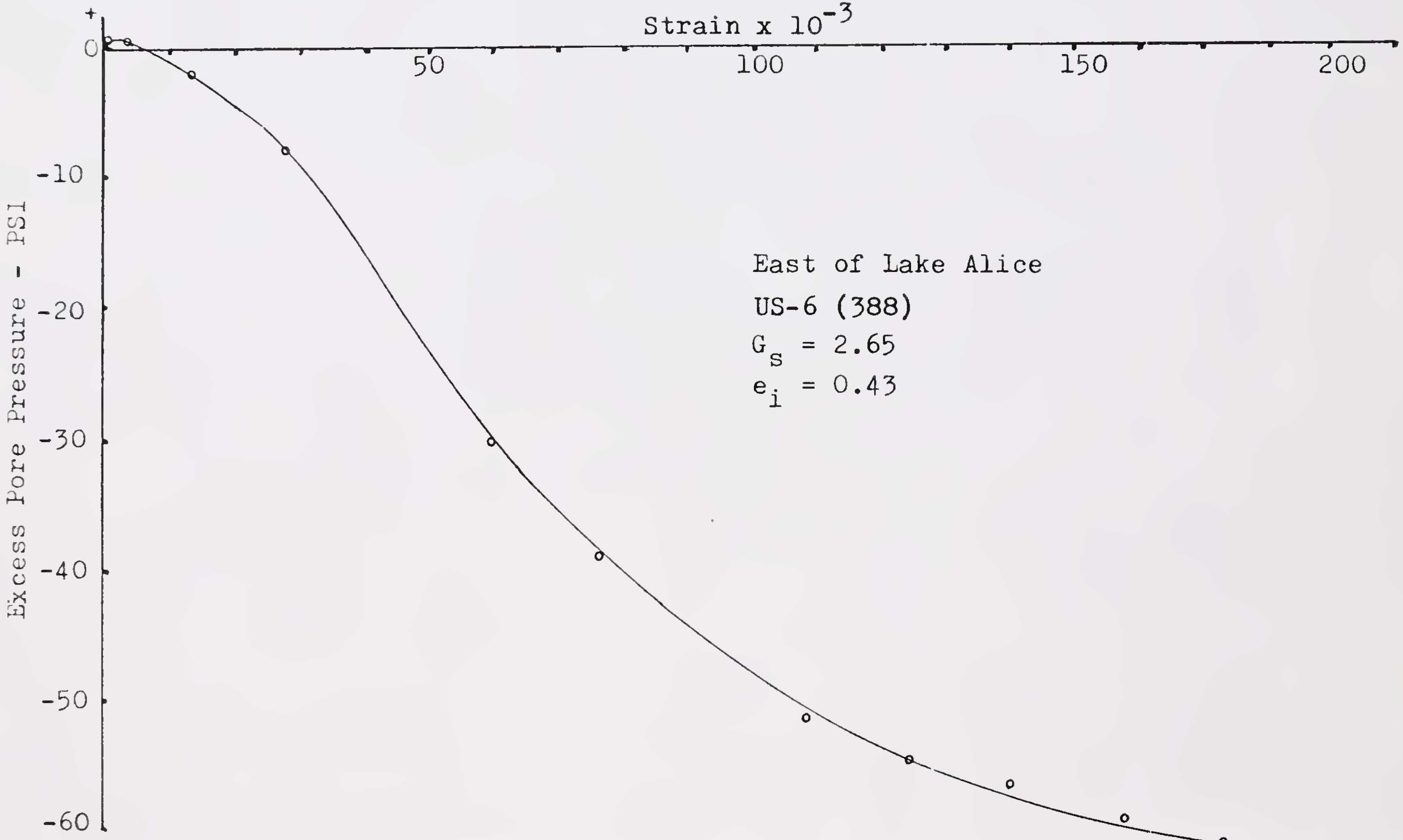


Figure E-17. Excess Pore Pressure - Axial Strain plot from US-6 (388)

APPENDIX F

STEREO PHOTOGRAPHY

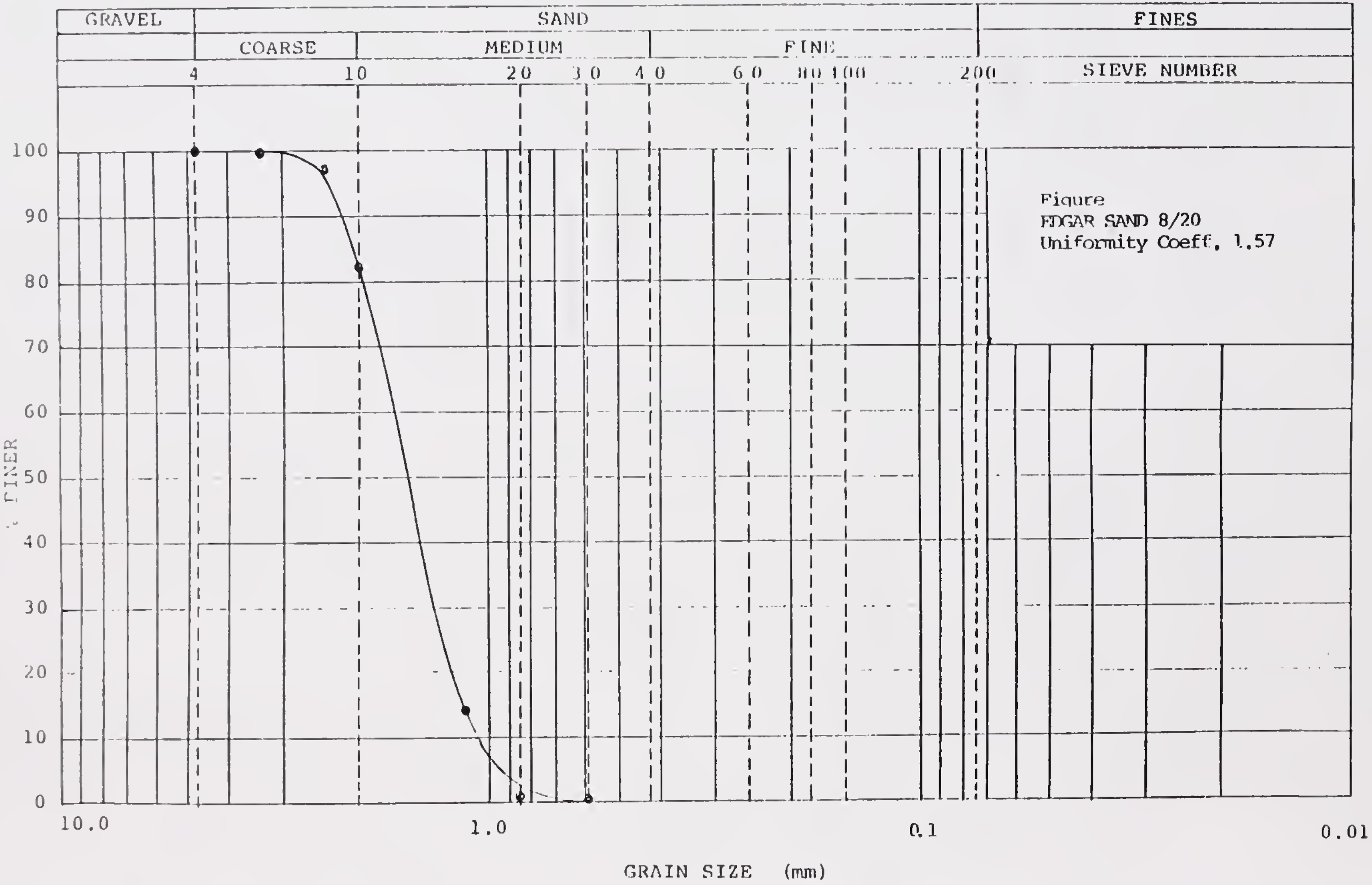


Figure F-1. Grain Size Curve for Edgar Sand 8/20

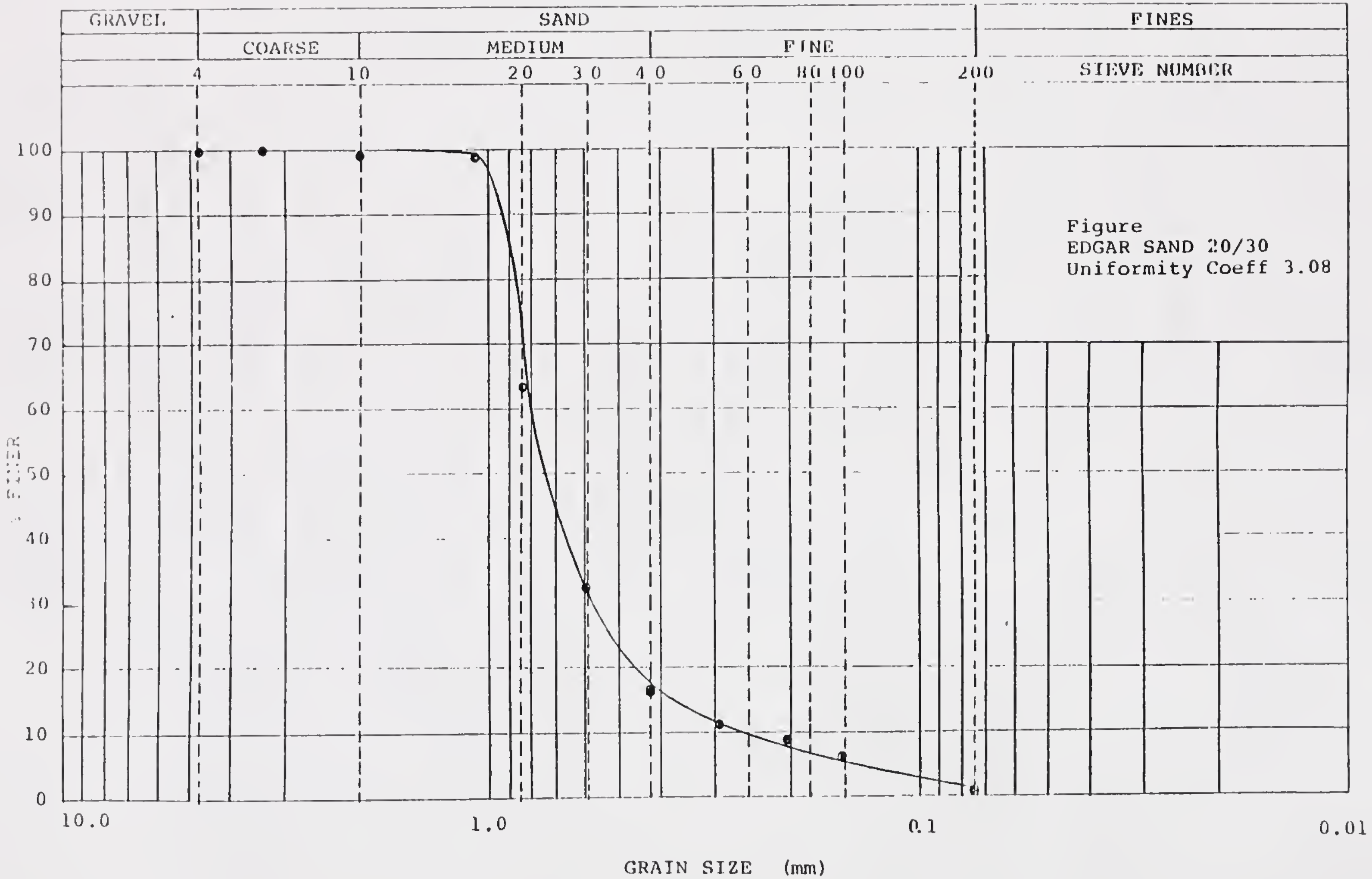


Figure F-2. Grain Size Curve for Edgar Sand 20/30

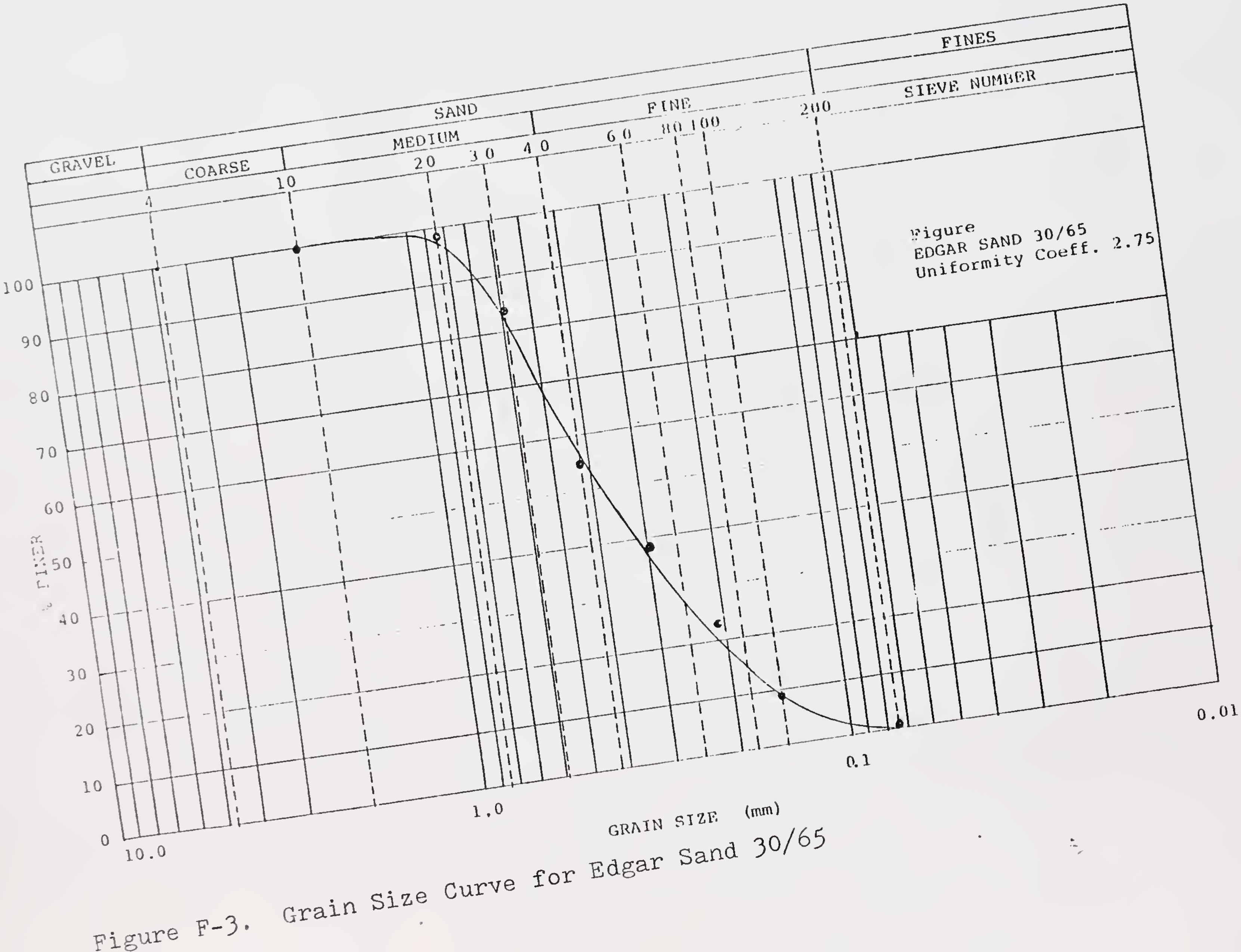


Figure F-3. Grain Size Curve for Edgar Sand 30/65

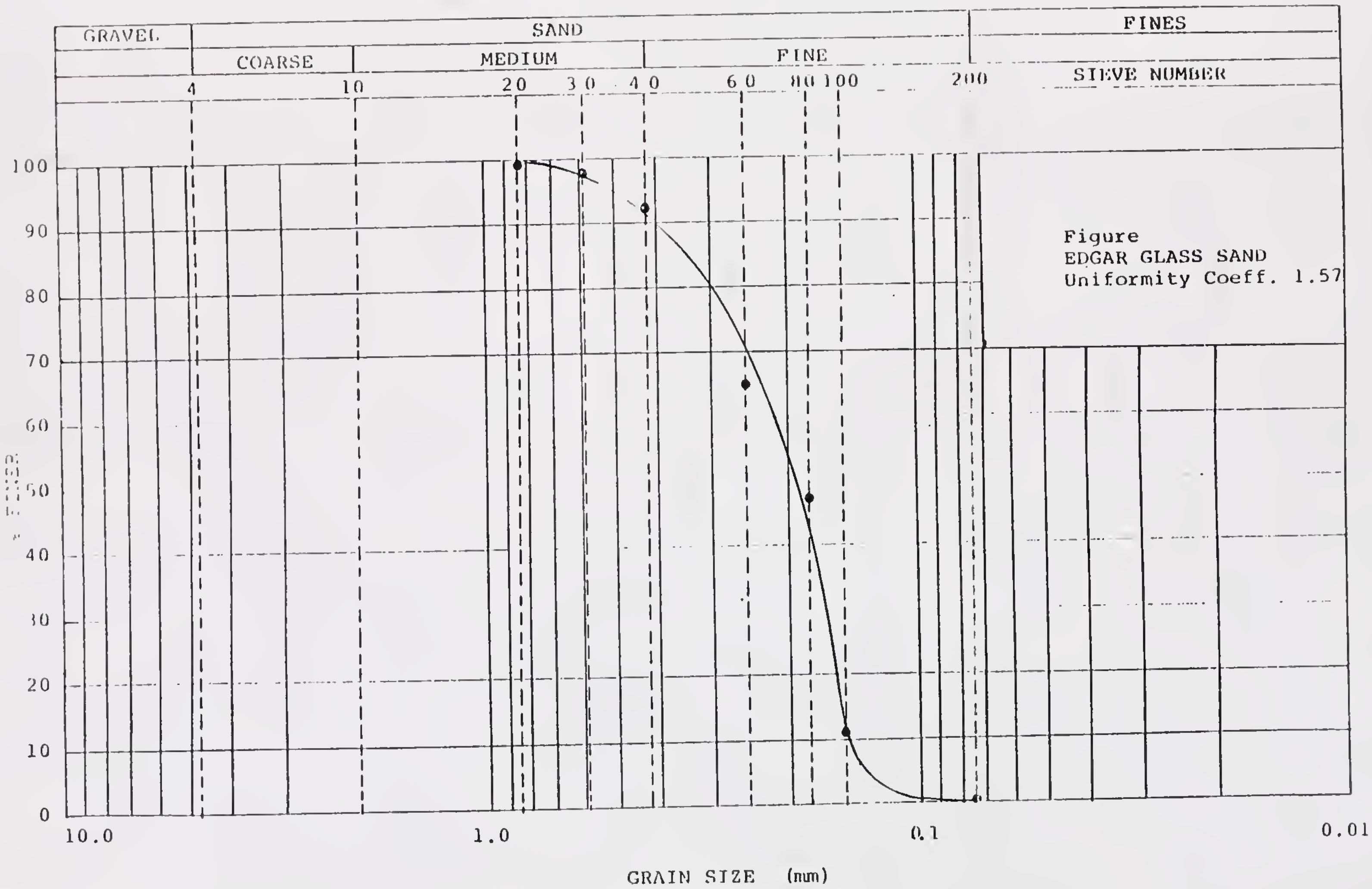


Figure F-4. Grain Size Curve for Edgar Glass Sand

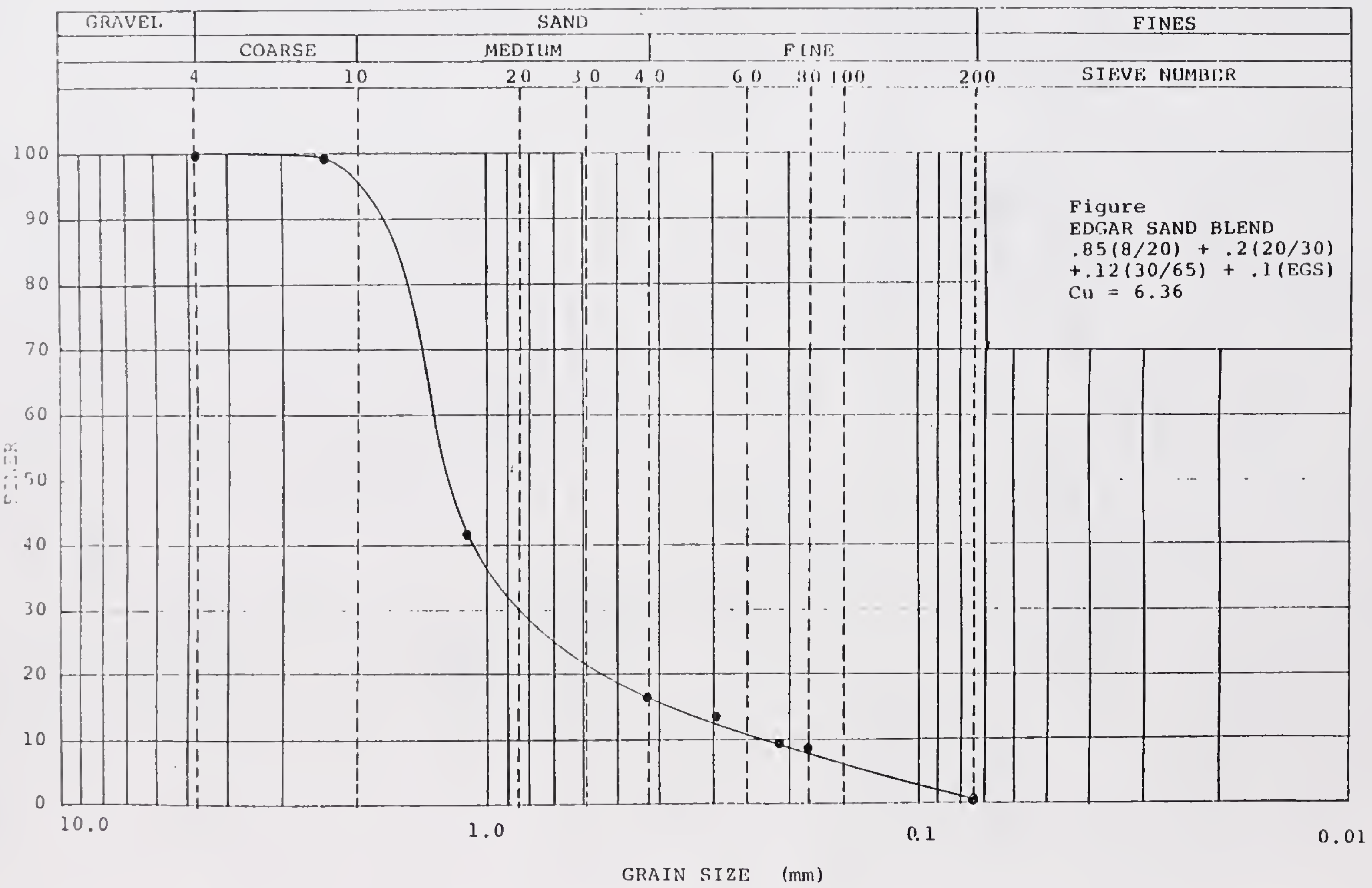


Figure F-5. Grain Size Curve for Edgar Sand Blend

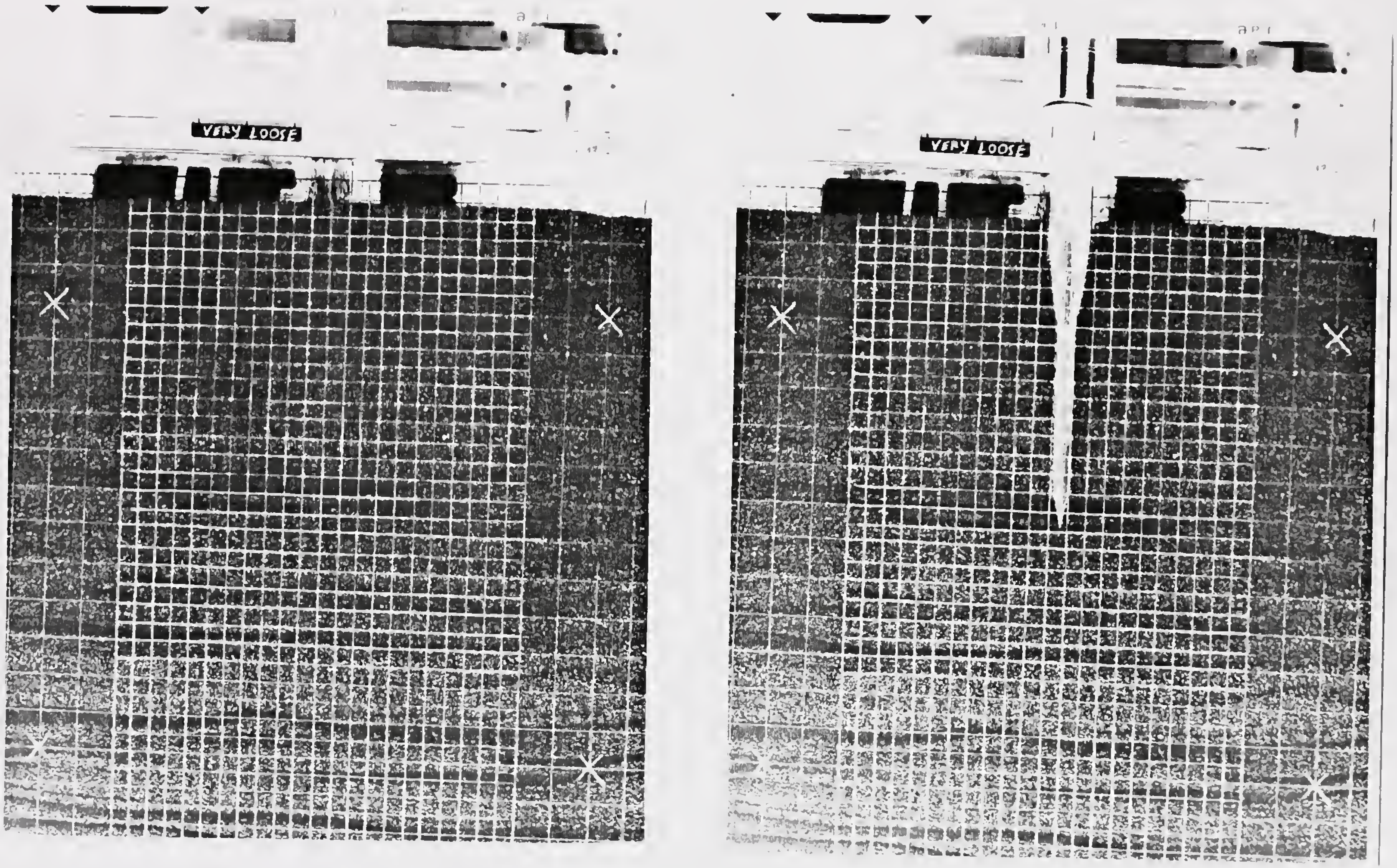


Figure F-6. Stereo Photographs - Very Loose Sand

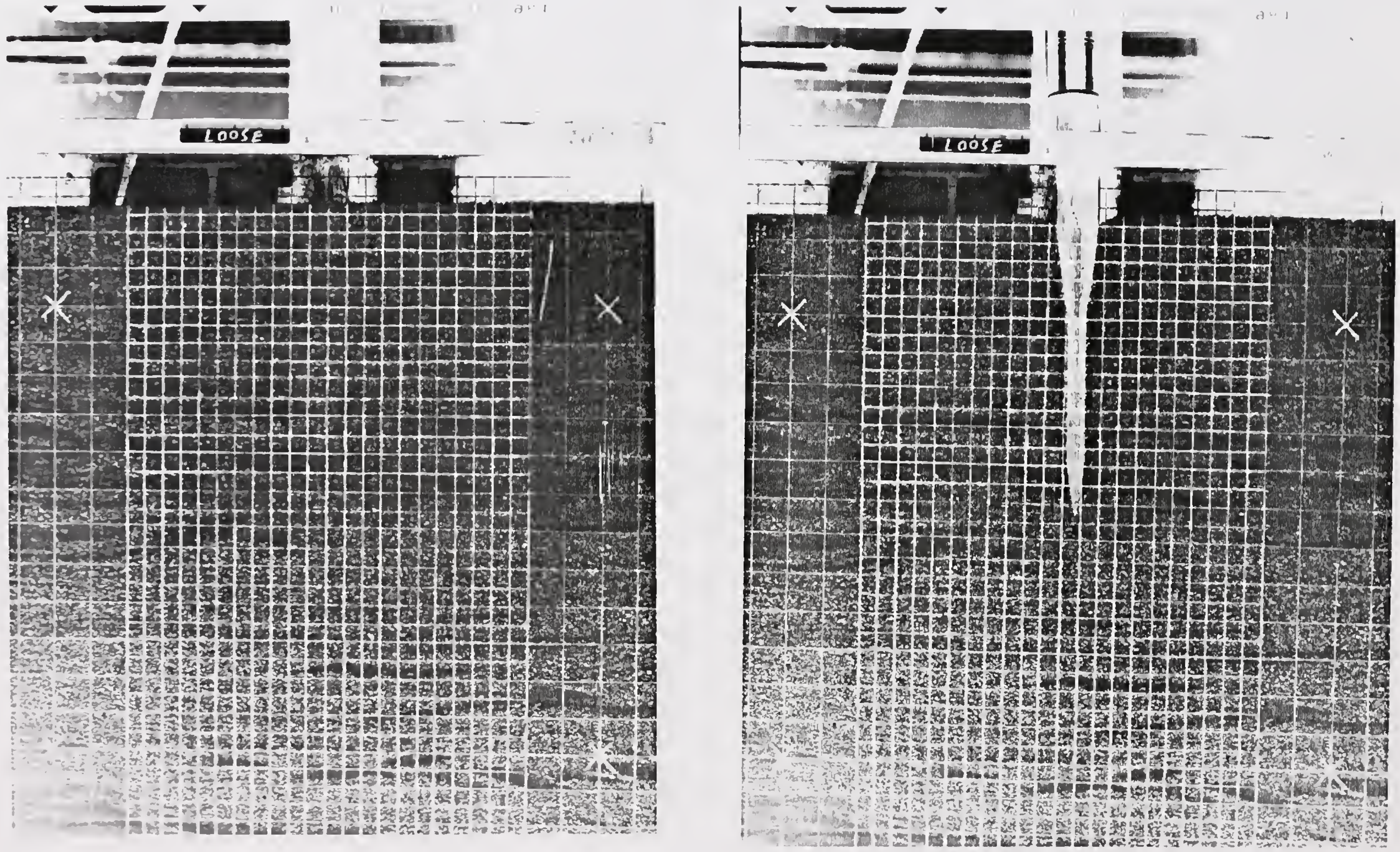


Figure F-7. Stereo Photographs - Loose Sand

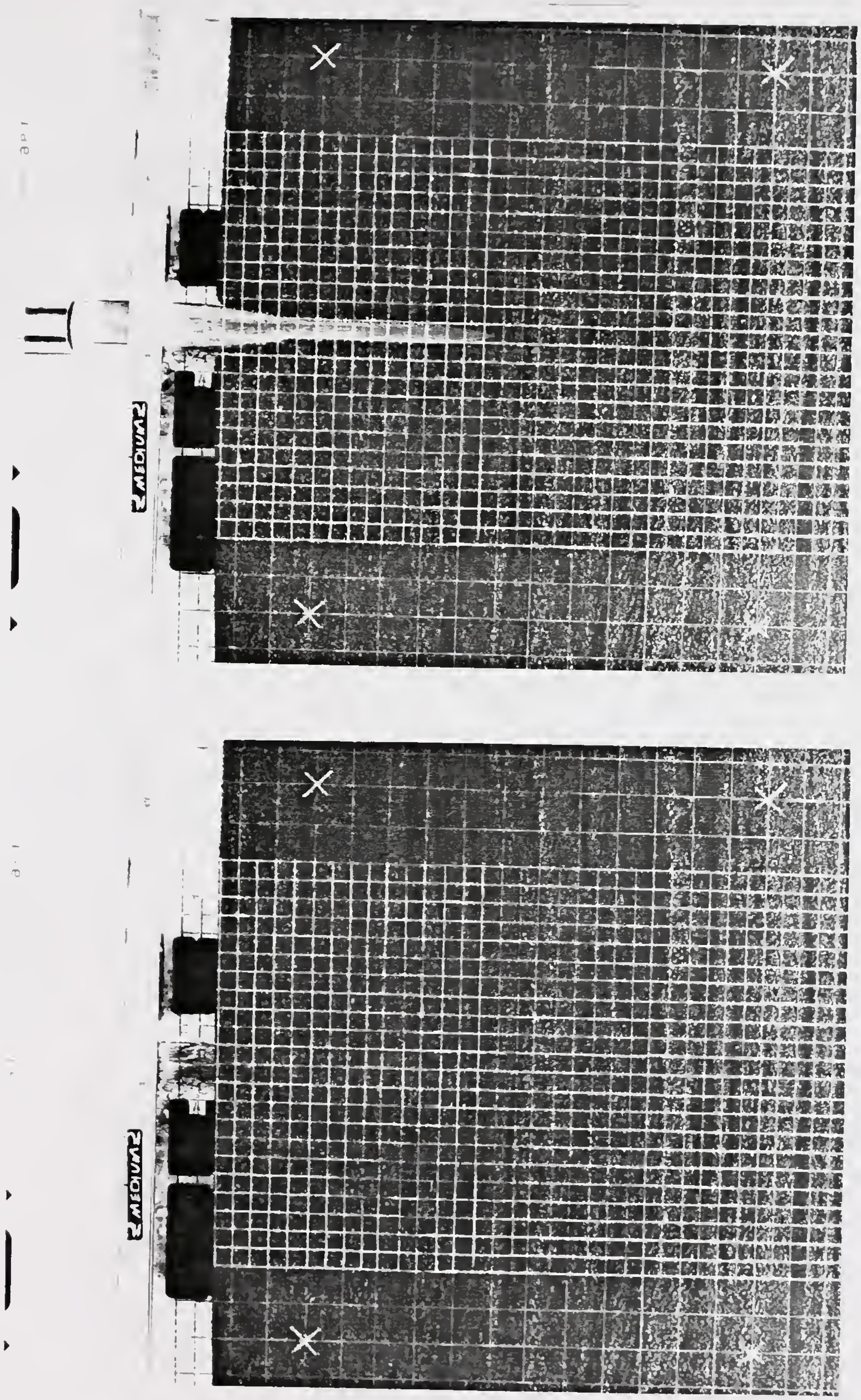


Figure F-8. Stereo Photographs - Medium Sand

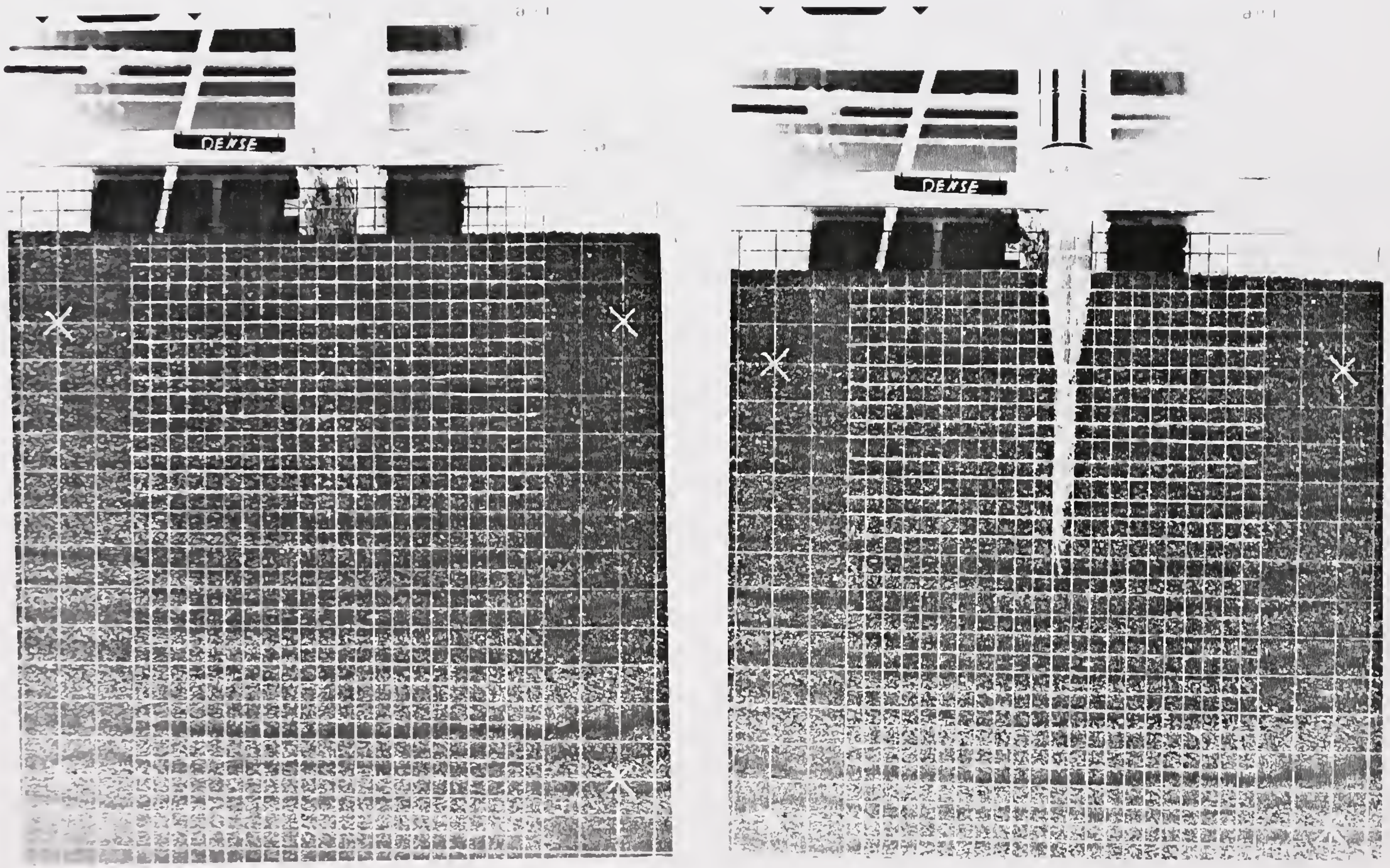


Figure F-9. Stereo Photographs - Dense Sand

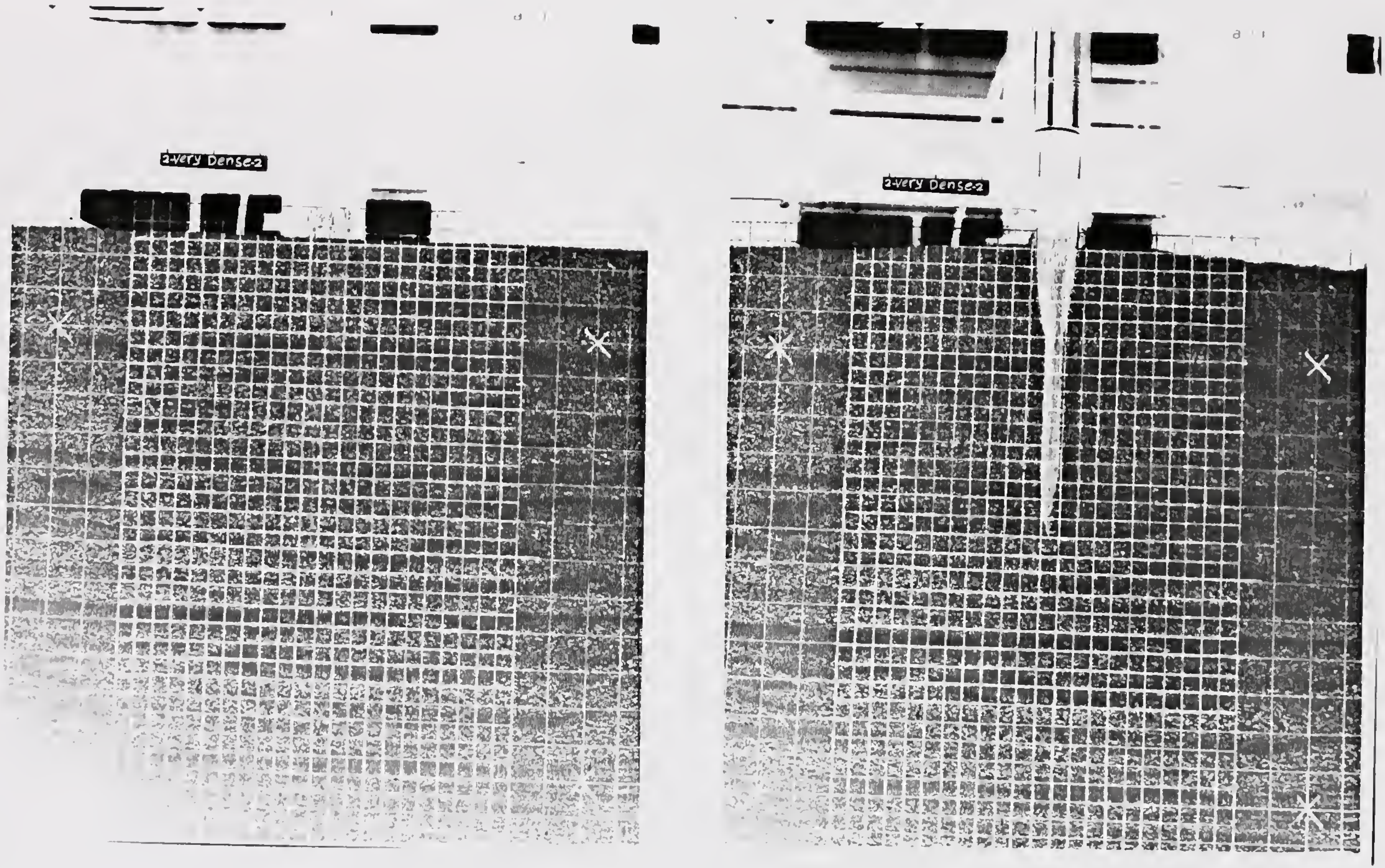


Figure F-10. Stereo Photographs - Very Dense Sand

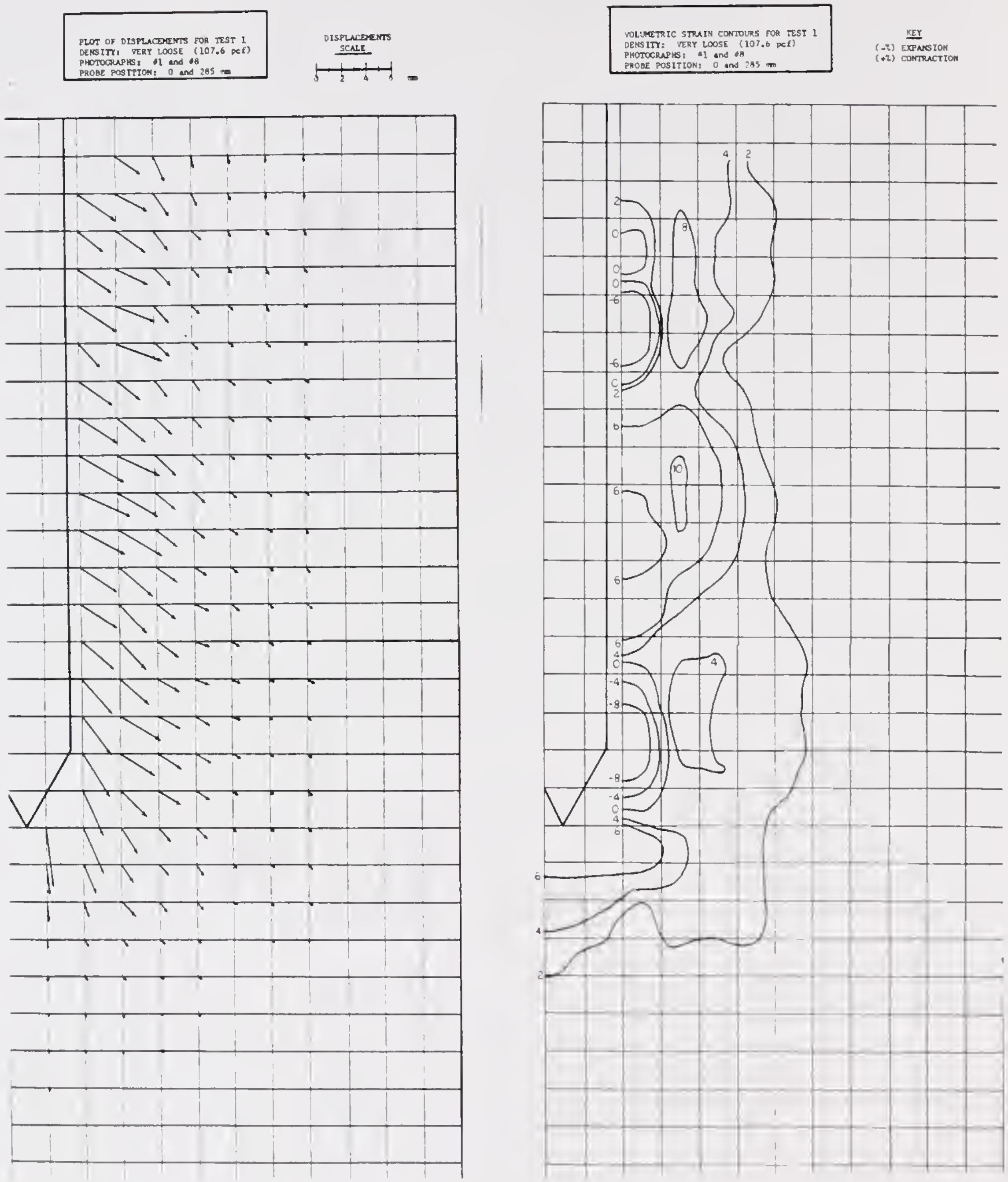


Figure F-11. Plots of Displacements and Volumetric Strain for Very Loose Sand (After Davidson, 1980)

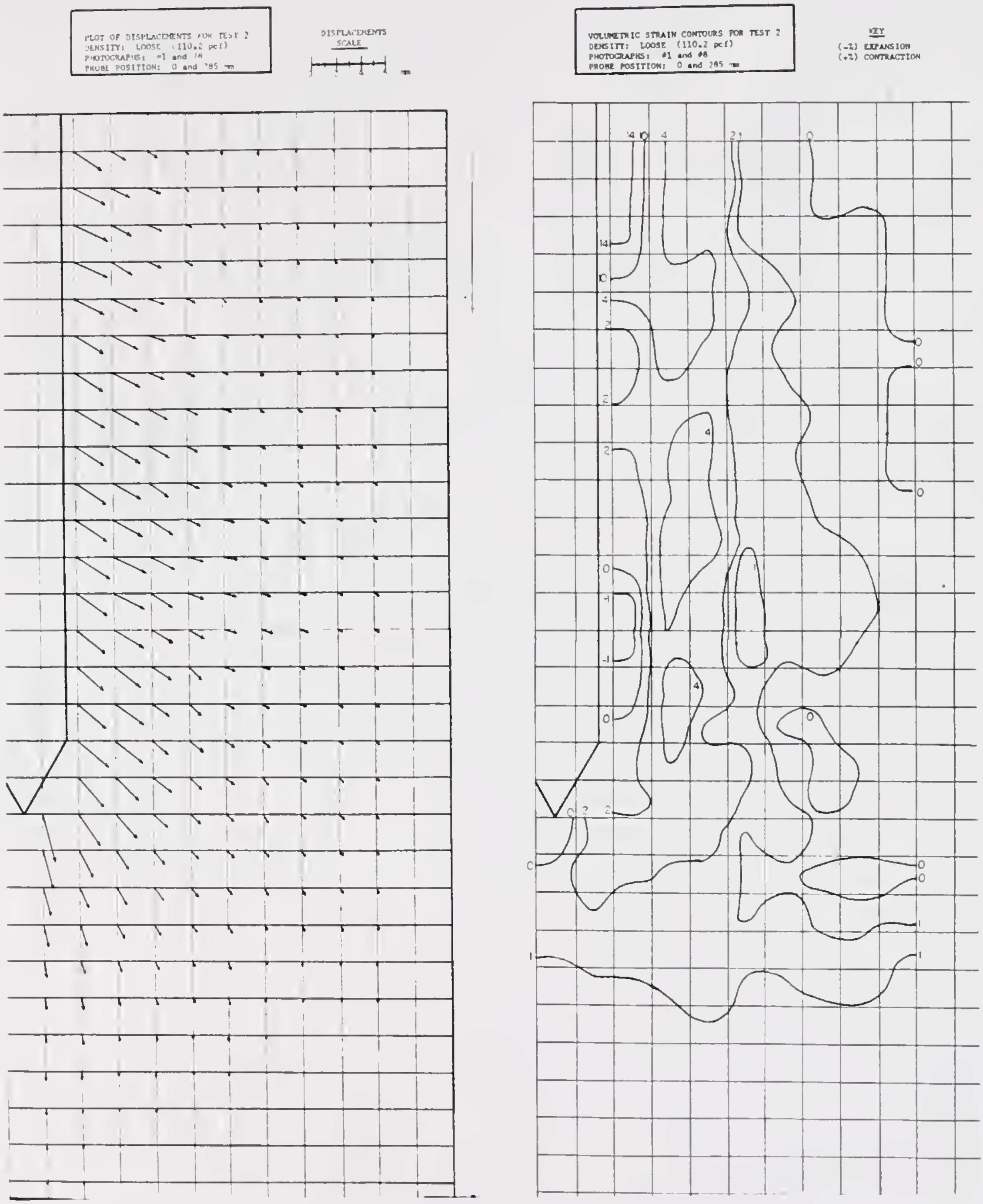


Figure F-12. Plots of Displacements and Volumetric Strain for Loose Sand (After Davidson, 1980)

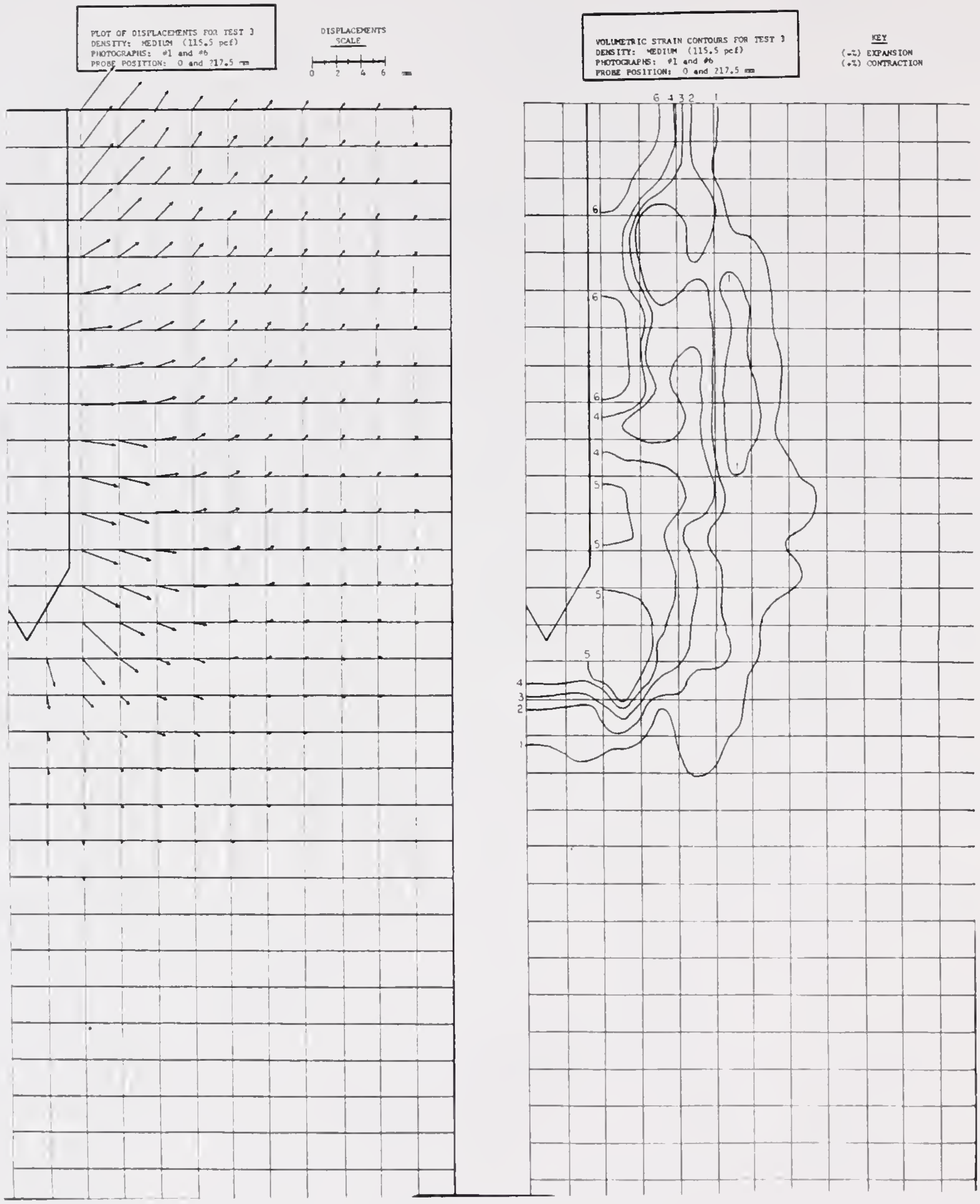


Figure F-13. Plots of Displacements and Volumetric Strain for Medium Sand (After Davidson, 1980)

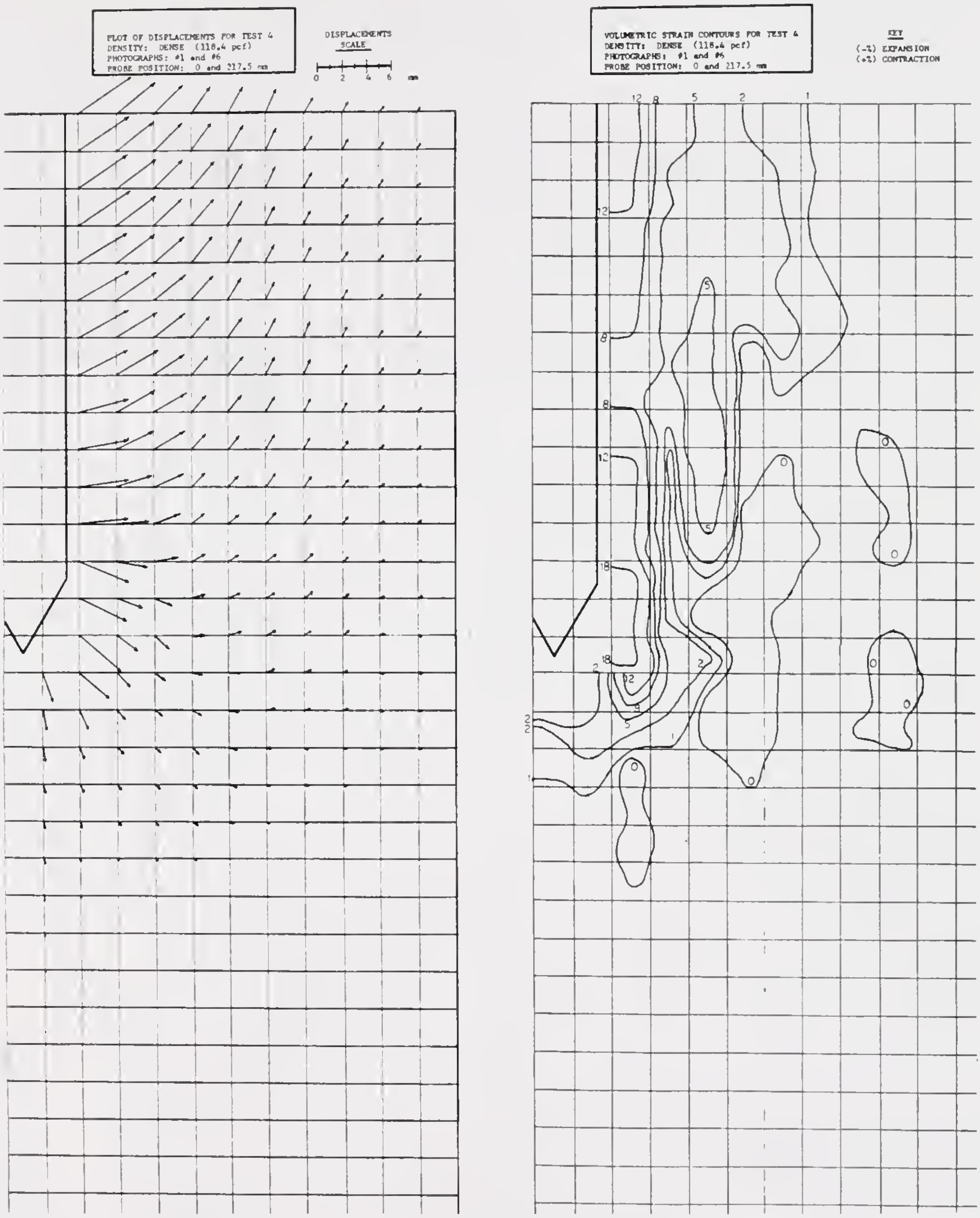

PLOT OF DISPLACEMENTS FOR TEST 4
DENSITY: DENSE (118.4 pcf)
PHOTOGRAPHS: #1 and #6
PROBE POSITION: 0 and 217.5 mm
DISPLACEMENTS SCALE
0
2
4
6
mm
VOLUMETRIC STRAIN CONTOURS FOR TEST 4
DENSITY: DENSE (118.4 pcf)
PHOTOGRAPHS: #1 and #6
PROBE POSITION: 0 and 217.5 mm
KEY
(-%) EXPANSION
(+%) CONTRACTION
12
8
5
2
1
0

Figure F-14. Plots of Displacements and Volumetric Strain for Dense Sand (After Davidson, 1980)

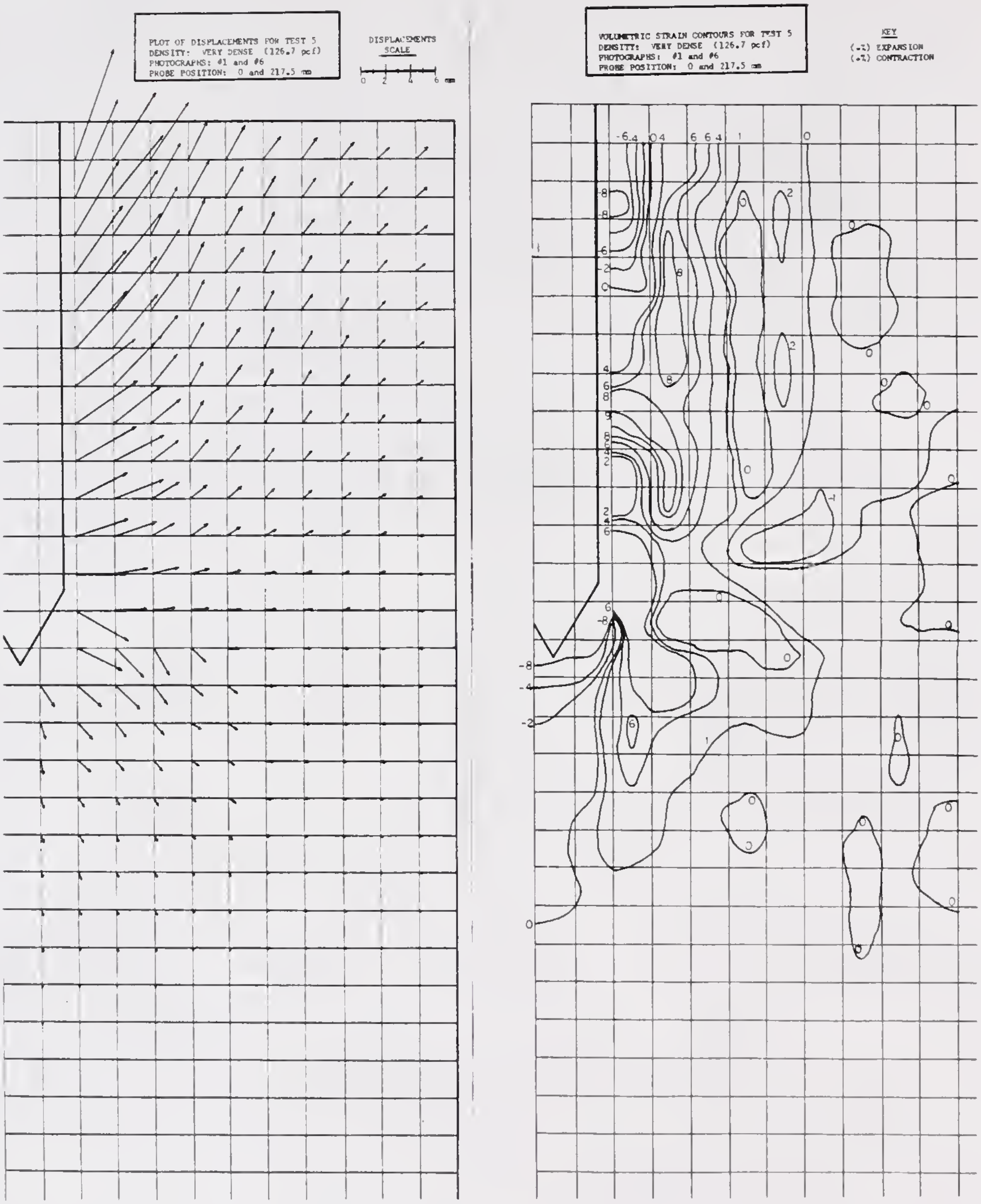


Figure F-15. Plots of Displacements and Volumetric Strain for Very Dense Sand (After Davidson, 1980)

```
 NODE          X-            Z-
 POINT       COORD.        COORD.


  676         46.50          1.50
  677         46.50          3.00
  678         46.50          4.50
  679         46.50          6.00
  680         46.50          7.50
  681         46.50          9.00
  682         46.50         10.50
  683         46.50         12.00
  684         46.50         13.50
  685         46.50         15.00
  686         46.50         16.50
  687         46.50         18.00
  688         46.50         19.50
  689         46.50         21.00
  690         46.50         22.50
  691         46.50         24.00
  692         46.50         25.50
  693         46.50         27.00
  694         46.50         28.50
  695         46.50         30.00
  696         46.50         31.50
  697         46.50         33.00
  698         46.50         34.50
  699         46.50         36.00
  700         46.50         37.50
  701         46.50         39.00
  702         46.50         40.50
  703         46.50         42.00
  704         46.50         43.50
  705         46.50         45.00
  706         46.50         46.50
  707         46.50         48.00
  708         46.50         49.50
  709         46.50         51.00
  710         46.50         52.50
  711         46.50         54.00
  712         46.50         55.50
  713         48.00          0.00
  714         48.00          1.50
  715         48.00          3.00
  716         48.00          4.50
  717         48.00          6.00
  718         48.00          7.50
  719         48.00          9.00
  720         48.00         10.50
  721         48.00         12.00
  722         48.00         13.50
  723         48.00         15.00
  724         48.00         16.50
  725         48.00         18.00
  726         48.00         19.50
  727         48.00         21.00
  728         48.00         22.50
  729         48.00         24.00
  730         48.00         25.50
```

Figure F-16. Typical Computer Output for Stereo Photography Analysis

ELEM. NO.	CORNER NODE POINTS A	B	C	D
676	698	699	737	736
677	699	700	738	737
678	700	701	739	738
679	701	702	740	739
680	702	703	741	740
681	703	704	742	741
682	704	705	743	742
683	705	706	744	743
684	706	707	745	744
685	707	708	746	745
686	708	709	747	746
687	709	710	748	747
688	710	711	749	748
689	711	712	750	749
690	713	714	752	751
691	714	715	753	752
692	715	716	754	753
693	716	717	755	754
694	717	718	756	755
695	718	719	757	756
696	719	720	758	757
697	720	721	759	758
698	721	722	760	759
699	722	723	761	760
700	723	724	762	761
701	724	725	763	762
702	725	726	764	763
703	726	727	765	764
704	727	728	766	765
705	728	729	767	766
706	729	730	768	767
707	730	731	769	768
708	731	732	770	769
709	732	733	771	770
710	733	734	772	771
711	734	735	773	772
712	735	736	774	773
713	736	737	775	774
714	737	738	776	775
715	738	739	777	776
716	739	740	778	777
717	740	741	779	778
718	741	742	780	779
719	742	743	781	780
720	743	744	782	781
721	744	745	783	782
722	745	746	784	783
723	746	747	785	784
724	747	748	786	785
725	748	749	787	786
726	749	750	788	787
727	751	752	790	789
728	752	753	791	790
729	753	754	792	791
730	754	755	793	792

Figure F-16. (Continued)

```
TEST #0  -  PHOTOGRAPHS #*  AND  #0

NODE    X-      Z-      X-DISP.   Z-DISP.   RESULT.
POINT  COORD.  COORD.    (MM)      (MM)      DISP.     ANGLE

 721   48.0    12.0    1.5582   -1.1317    1.9258   35.9915
 722   48.0    13.5    1.6074   -0.8857    1.8353   28.8557
 723   48.0    15.0    1.5746   -0.7873    1.7604   26.5650
 724   48.0    16.5    1.5746   -0.7381    1.7390   25.1148
 725   48.0    18.0    1.5418   -0.6561    1.6756   23.0513
 726   48.0    19.5    1.5582   -0.5741    1.6606   20.2249
 727   48.0    21.0    1.5254   -0.4921    1.6028   17.8787
 728   48.0    22.5    1.5582   -0.3608    1.5994   13.0387
 729   48.0    24.0    1.3286   -0.1640    1.3386    7.0379
 730   48.0    25.5    1.1481   -0.0656    1.1500    3.2705
 731   48.0    27.0    0.9513    1.0169    1.3925   46.9091
 732   48.0    28.5    0.4265    0.5577    0.7020   52.5946
 733   48.0    30.0    0.2952    0.3936    0.4921   53.1301
 734   48.0    31.5    0.2132    0.2296    0.3134   47.1211
 735   48.0    33.0    0.1640    0.1312    0.2100   38.6598
 736   48.0    34.5    0.0000    0.0000    0.0000    0.0000
 737   48.0    36.0    0.0000    0.0000    0.0000    0.0000
 738   48.0    37.5    0.0000    0.0000    0.0000    0.0000
 739   48.0    39.0    0.0000    0.0000    0.0000    0.0000
 740   48.0    40.5    0.0000    0.0000    0.0000    0.0000
 741   48.0    42.0    0.0000    0.0000    0.0000    0.0000
 742   48.0    43.5    0.0000    0.0000    0.0000    0.0000
 743   48.0    45.0    0.0000    0.0000    0.0000    0.0000
 744   48.0    46.5    0.0000    0.0000    0.0000    0.0000
 745   48.0    48.0    0.0000    0.0000    0.0000    0.0000
 746   48.0    49.5    0.0000    0.0000    0.0000    0.0000
 747   48.0    51.0    0.0000    0.0000    0.0000    0.0000
 748   48.0    52.5    0.0000    0.0000    0.0000    0.0000
 749   48.0    54.0    0.0000    0.0000    0.0000    0.0000
 750   48.0    55.5    0.0000    0.0000    0.0000    0.0000
 751   49.5     0.0    0.0000    0.0000    0.0000    0.0000
 752   49.5     1.5    0.0000    0.0000    0.0000    0.0000
 753   49.5     3.0    0.0000    0.0000    0.0000    0.0000
 754   49.5     4.5    2.0502   -2.2963    3.0784   48.2397
 755   49.5     6.0    1.9682   -2.2471    2.9872   48.7845
 756   49.5     7.5    1.8534   -2.1427    2.8376   49.2191
 757   49.5     9.0    1.4926   -1.3286    1.9982   41.6726
 758   49.5    10.5    1.2794   -1.0497    1.6549   39.3693
 759   49.5    12.0    1.2794   -0.9841    1.6141   37.5686
 760   49.5    13.5    1.2301   -0.7381    1.4346   30.9638
 761   49.5    15.0    1.1645   -0.6889    1.3530   30.6064
 762   49.5    16.5    1.2466   -0.6561    1.4087   27.7585
 763   49.5    18.0    1.2137   -0.5577    1.3357   24.6769
 764   49.5    19.5    1.2137   -0.5249    1.3224   23.3852
 765   49.5    21.0    1.2630   -0.4593    1.3439   19.9831
 766   49.5    22.5    1.1645   -0.3280    1.2099   15.7320
 767   49.5    24.0    0.9841   -0.1476    0.9951    8.5308
 768   49.5    25.5    0.8365    0.1640    0.8524   11.0937
 769   49.5    27.0    0.7053    0.5741    0.9094   39.1440
 770   49.5    28.5    0.4100    0.4100    0.5799   45.0000
 771   49.5    30.0    0.2296    0.2952    0.3740   52.1250
 772   49.5    31.5    0.2296    0.1640    0.2822   35.5377
```

Figure F-16. (Continued)

```
        TEST #0  -  PHOTOGRAPHS #8  AND  #0


ELEM.  X-       X - STRAIN     Z-        Z - STRAIN
 NO. COORD.    DA        BC   COORD.    AB        CD


 721  48.75   0.000    0.000  47.25    0.000    0.000
 722  48.75   0.000    0.000  48.75    0.000    0.000
 723  48.75   0.000    0.000  50.25    0.000    0.000
 724  48.75   0.000    0.000  51.75    0.000    0.000
 725  48.75   0.000    0.000  53.25    0.000    0.000
 726  48.75   0.000    0.000  54.75    0.000    0.000
 727  50.25   0.000    0.000   0.75    0.000    0.000
 728  50.25   0.000    0.000   2.25    0.000    0.000
 729  50.25   0.000    6.014   3.75   15.309   13.340
 730  50.25   6.014    5.247   5.25   -0.328   -0.109
 731  50.25   5.247    5.249   6.75   -0.656    0.875
 732  50.25   5.249    4.702   8.25   -5.467   -5.905
 733  50.25   4.702    4.155   9.75   -1.859   -1.640
 734  50.25   4.155    4.046  11.25   -0.437   -0.765
 735  50.25   4.046    4.046  12.75   -1.640   -1.312
 736  50.25   4.046    3.498  14.25   -0.328   -0.656
 737  50.25   3.498    3.339  15.75   -0.219    0.000
 738  50.25   3.389    3.499  17.25   -0.656   -0.547
 739  50.25   3.499    3.499  18.75   -0.219   -0.109
 740  50.25   3.499    4.155  20.25   -0.437   -0.437
 741  50.25   4.155    3.389  21.75   -0.875   -0.765
 742  50.25   3.389    2.733  23.25   -1.203   -0.875
 743  50.25   2.733    2.188  24.75   -2.078   -2.187
 744  50.25   2.188    1.421  26.25   -2.733   -2.186
 745  50.25   1.421    0.547  27.75    1.094    1.094
 746  50.25   0.547   -0.218  29.25    0.765    0.983
 747  50.25  -0.218    0.218  30.75    0.876    0.438
 748  50.25   0.218    1.092  32.25    0.547    0.765
 749  50.25   1.092    0.000  33.75    0.545    0.000
 750  50.25   0.000    0.000  35.25    0.000    0.000
 751  50.25   0.000    0.000  36.75    0.000    0.000
 752  50.25   0.000    0.000  38.25    0.000    0.000
 753  50.25   0.000    0.000  39.75    0.000    0.000
 754  50.25   0.000    0.000  41.25    0.000    0.000
 755  50.25   0.000    0.000  42.75    0.000    0.000
 756  50.25   0.000    0.000  44.25    0.000    0.000
 757  50.25   0.000    0.000  45.75    0.000    0.000
 758  50.25   0.000    0.000  47.25    0.000    0.000
 759  50.25   0.000    0.000  48.75    0.000    0.000
 760  50.25   0.000    0.000  50.25    0.000    0.000
 761  50.25   0.000    0.000  51.75    0.000    0.000
 762  50.25   0.000    0.000  53.25    0.000    0.000
 763  50.25   0.000    0.000  54.75    0.000    0.000
 764  51.75   0.000    0.000   0.75    0.000    0.000
 765  51.75   0.000    0.000   2.25    0.000    0.000
 766  51.75   0.000    2.952   3.75   13.340   11.481
 767  51.75   2.952    3.499   5.25   -0.109   -0.219
 768  51.75   3.499    3.717   6.75    0.875   -2.515
 769  51.75   3.717    2.515   8.25   -5.905   -0.875
 770  51.75   2.515    2.078   9.75   -1.640   -1.968
 771  51.75   2.078    1.968  11.25   -0.765   -0.656
 772  51.75   1.968    1.748  12.75   -1.312   -0.656
 773  51.75   1.748    2.078  14.25   -0.656   -0.547
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Figure F-16. (Continued)

TEST #0 - PHOTOGRAPHS #8 AND #0

ELEM. NO.	X- COORD.	Z- COORD.	SHEAR STRAIN
721	48.00	46.50	0.000
722	48.00	48.00	0.000
723	48.00	49.50	0.000
724	48.00	51.00	0.000
725	48.00	52.50	0.000
726	48.00	54.00	0.000
727	49.50	0.00	0.000
728	49.50	1.50	0.000
729	49.50	3.00	0.137
730	49.50	4.50	0.014
731	49.50	6.00	0.010
732	49.50	7.50	-0.022
733	49.50	9.00	-0.008
734	49.50	10.50	0.004
735	49.50	12.00	0.004
736	49.50	13.50	0.000
737	49.50	15.00	0.013
738	49.50	16.50	0.003
739	49.50	18.00	0.004
740	49.50	19.50	0.007
741	49.50	21.00	-0.003
742	49.50	22.50	-0.010
743	49.50	24.00	-0.011
744	49.50	25.50	-0.009
745	49.50	27.00	-0.025
746	49.50	28.50	-0.017
747	49.50	30.00	-0.008
748	49.50	31.50	-0.008
749	49.50	33.00	-0.016
750	49.50	34.50	0.000
751	49.50	36.00	0.000
752	49.50	37.50	0.000
753	49.50	39.00	0.000
754	49.50	40.50	0.000
755	49.50	42.00	0.000
756	49.50	43.50	0.000
757	49.50	45.00	0.000
758	49.50	46.50	0.000
759	49.50	48.00	0.000
760	49.50	49.50	0.000
761	49.50	51.00	0.000
762	49.50	52.50	0.000
763	49.50	54.00	0.000
764	51.00	0.00	0.000
765	51.00	1.50	0.000
766	51.00	3.00	0.077
767	51.00	4.50	0.021
768	51.00	6.00	0.012
769	51.00	7.50	0.035
770	51.00	9.00	-0.005
771	51.00	10.50	0.008
772	51.00	12.00	0.002
773	51 00	13.50	0.000

Figure F-16. (Continued)

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TEST #0  -  PHOTOGRAPHS #*  AND  #G

ELEM.    X-      Z-       DENSITY (PCF)       VOLUMETRIC
NO.    COORD.  COORD.   PREVIOUS     NEW        STRAIN


721    48.75   47.25     118.55    118.55        0.00
722    48.75   48.75     118.55    118.55        0.00
723    48.75   50.25     118.55    118.55        0.00
724    48.75   51.75     118.55    118.55        0.00
725    48.75   53.25     118.55    118.55        0.00
726    48.75   54.75     118.55    118.55        0.00
727    50.25    0.75     118.55    118.55        0.00
728    50.25    2.25     118.55    118.55        0.00
729    50.25    3.75     118.55    142.54       17.01
730    50.25    5.25     118.55    125.32        5.40
731    50.25    6.75     118.55    125.17        5.29
732    50.25    8.25     118.55    117.97       -0.49
733    50.25    9.75     118.55    121.86        2.71
734    50.25   11.25     118.55    122.85        3.50
735    50.25   12.75     118.55    121.71        2.60
736    50.25   14.25     118.55    122.59        3.29
737    50.25   15.75     118.55    122.66        3.35
738    50.25   17.25     118.55    122.04        2.86
739    50.25   18.75     118.55    122.64        3.33
740    50.25   20.25     118.55    122.74        3.42
741    50.25   21.75     118.55    122.16        2.96
742    50.25   23.25     118.55    121.04        2.06
743    50.25   24.75     118.55    119.01        0.39
744    50.25   26.25     118.55    117.81       -0.63
745    50.25   27.75     118.55    121.07        2.08
746    50.25   29.25     118.55    119.80        1.04
747    50.25   30.75     118.55    119.31        0.64
748    50.25   32.25     118.55    120.14        1.32
749    50.25   33.75     118.55    119.55        0.83
750    50.25   35.25     118.55    118.55        0.00
751    50.25   36.75     118.55    118.55        0.00
752    50.25   38.25     118.55    118.55        0.00
753    50.25   39.75     118.55    118.55        0.00
754    50.25   41.25     118.55    118.55        0.00
755    50.25   42.75     118.55    118.55        0.00
756    50.25   44.25     118.55    118.55        0.00
757    50.25   45.75     118.55    118.55        0.00
758    50.25   47.25     118.55    118.55        0.00
759    50.25   48.75     118.55    118.55        0.00
760    50.25   50.25     118.55    118.55        0.00
761    50.25   51.75     118.55    118.55        0.00
762    50.25   53.25     118.55    118.55        0.00
763    50.25   54.75     118.55    118.55        0.00
764    51.75    0.75     118.55    118.55        0.00
765    51.75    2.25     118.55    118.55        0.00
766    51.75    3.75     118.55    137.47       13:76
767    51.75    5.25     118.55    122.29        3.06
768    51.75    6.75     118.55    121.90        2.75
769    51.75    8.25     118.55    118.27       -0.24
770    51.75    9.75     118.55    119.13        0.49
771    51.75   11.25     118.55    120.10        1.29
772    51.75   12.75     118.55    119.59        0.86
773    51.75   14.25     118.55    120.14        1.32
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Figure F-16. (Continued)

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BIOGRAPHICAL SKETCH

Alireza Boghrat was born April 20, 1950, in Khash, Iran. He graduated from high school in 1968, and in 1969 came to the United States to start college. Boghrat completed two years at Tallahassee Junior College and entered the University of Florida in 1972. He received a bachelor's degree in civil engineering in 1974 and the degree of Master of Engineering (ME) in 1975. Boghrat returned to Iran and worked as an instructor from 1976 to 1979 at the Iran University of Science and Technology (IUST). In 1979 he received a scholarship from IUST and returned to the United States to continue his education toward a Ph.D. in geotechnical engineering. Mr. Boghrat was married on June 26, 1971, to the former Shirley Lucas and is the proud father of a boy and girl.

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

John L. Davidson, Chairman
Associate Professor of Civil Engineering

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

James H Schaub

James H. Schaub
Professor of Civil Engineering

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Frank C Townsend

Frank C. Townsend
Professor of Civil Engineering

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

Walter H. Zimpfer

Walter H. Zimpfer
Associate Professor of Civil
Engineering

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

James L. Eades
Associate Professor of Geology

This dissertation was submitted to the Graduate Faculty of the College of Engineering and to the Graduate Council, and was accepted as partial fulfillment of the requirements for the degree of Doctor of Philosophy.

December 1982

Wayne H. Chen

Dean, College of Engineering

Dean, Graduate School